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Woodward-Clyde Consultants

GEOTECHNICAL INVESTIGATION
FLOOD CONTROL IMPROVEMENTS
COYOTE CREEK
HIGHWAY 237 TO SAN FRANCISCO BAY

Santa Clara County, California

Prepared For

SANTA CLARA VALLEY WATER DISTRICT
Mr. David M. McNeely
Design Division Engineer
5750 Almaden Expressway
San Jose, California 95118

1180 Coleman Avenue
San Jose, California 95110
408-297-9585

Woodward-Clyde Consultants

April 15, 1985

Project: 15945-W

Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118

Attention: Mr. David M. McNeely
Design Division Engineer

Gentlemen:

We are pleased to submit our report presenting the results of our geotechnical investigation made along the alignment of the proposed flood control improvements to the Coyote Creek from Highway 237 to the San Francisco Bay.

The accompanying report presents our conclusions and recommendations regarding the geotechnical engineering aspects of the project design. The opinions and recommendations have been based upon the results of our field explorations and laboratory testing, a review of existing available data, engineering judgment and experience. During the course of our investigation, meetings were held with the District design team to present and discuss our findings and conclusions as they were developed. Geotechnical recommendations were developed in consultation with the District to accommodate the various hydrologic and civil engineering design constraints.

The report was initially submitted as a draft for the District's review and comment. The results of that review and comment have been incorporated into this final report.

It has been a pleasure to provide geotechnical design services on this important and challenging project. If any questions should arise regarding the contents of this report, or when we can be of further service, please call us.

Sincerely,

Paul J. Boddie

Paul J. Boddie
Senior Project Engineer

William A. Bischoff

William A. Bischoff
Vice President

PJB/WAB/rm

Consulting Engineers, Geologists
and Environmental Scientists

Offices in Other Principal Cities

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GEOTECHNICAL INVESTIGATION
FLOOD CONTROL IMPROVEMENTS ON COYOTE CREEK
HIGHWAY 237 TO SAN FRANCISCO BAY
Santa Clara County, California

INTRODUCTION

This report presents the results of a geotechnical investigation of the subsurface conditions along the alignment of the proposed flood control improvements on the Coyote Creek extending from Highway 237 to the San Francisco Bay. The improvements will be comprised of a system of flood plain levees in combination with a new bypass channel which will divert flood waters through Coyote Slough along the south side of Newby Island. At the present time Coyote Creek flows around the north side of the island.

For design purposes, the Santa Clara Valley Water District has divided the alignment into two major reaches. Reach 1 will extend from the northerly limit of the project near the confluence of Coyote Creek and Coyote Slough upstream to the Milpitas Sanitation District facility. It is within Reach 1 that the proposed bypass channel is to be developed. Reach 1 is subdivided further into Reach 1A, which traverses an evaporation pond of the Leslie Salt Company; and Reach 1B which passes between the extension of Dixon Landing Road and the sludge lagoons of the San Jose-Santa Clara Water Pollution Control Plant. Reach 2 then extends from the Milpitas Sanitation District facility upstream to about 400 feet south of the existing Highway 237 bridge. The general project alignment and limits of each reach are shown on the attached Site and Exploration Plans, Figures 1a through 1c.

Report Format

For convenience, this report also has been divided into sections corresponding to the individual reaches of the alignment. Specific geotechnical recommendations for each reach are presented within these individual report sections together with descriptions of the subsurface conditions and related geotechnical considerations. Recommendations which are applicable to all portions of the proposed improvements, such as earthwork considerations, are presented in the section entitled "General Recommendations". The section entitled "Assessment of Levee Stability During Earthquakes" presents the results of a study of seismic stability and estimated deformation of the proposed levees for selected design earthquakes together with a discussion of remedial measures which might be used to increase the seismic levee stability. The conclusions regarding the seismic stability of the levees were considered in the development of recommendations for each project reach.

Existing Subsurface Data

Prior to laying out the field exploration program for this investigation, existing subsurface data along the project alignment was collected and reviewed. The list of data sources which were consulted is given below:

- Woodward-Lundgren & Associates preliminary soil investigation report for flood control improvements along Coyote Creek dated February 10, 1972.
- A soil investigation report prepared by Dames & Moore for the sludge lagoon additions (1970) to the San Jose-Santa Clara Water Pollution Control Plant dated August 17, 1971, and furnished by the City of San Jose.

- A soil investigation report by Peter Kaldveer and Associates, Inc. for proposed sludge drying lagoon at the San Jose-Santa Clara Water Pollution Control Plant dated May 26, 1983, and furnished by the City of San Jose.
- Published U.S.G.S. and County of Santa Clara data.

The locations of borings from these available data sources are shown on Figures 1a through 1c. However, logs of the borings are not included in this report.

Field Exploration and Testing

To supplement the available data and obtain more detailed design level information, 40 additional exploratory borings and 10 exploratory pits were made along the project alignment at the locations shown on Figures 1a through 1c. A detailed description of the field exploration and laboratory testing program is presented in Appendix A. Logs of Borings are presented on Figures A-1 through A-40 and Logs of Exploratory Pits 1 through 9 are presented on Figures A-41 through A-49. Laboratory test results for samples recovered from these explorations are presented on the Logs of Borings and Exploratory Pits as well as on Figures A-50 through A-66 and on Tables A-1 and A-2. The results of insitu field vane shear tests are summarized on Table A-3 and the results of a geologic reconnaissance of nearby quarries are presented on Table A-4.

PROPOSED BYPASS CHANNEL DESCRIPTION

Within the Reach 1A portion of the alignment, it is planned to provide a bypass channel with increased flow capacity through the Coyote Slough area. This will be accomplished by constructing a new levee across the existing salt evaporation pond. The resulting bypass channel will have a width in excess of 900 feet between the new levee and existing levee along the south side of Newby Island.

The new levee will begin near the existing salt pond syphon and will extend approximately 7,000 feet to the east side of the pond as shown on Figure 1a. Portions of the new levee will follow the alignment of the existing salt pond levee; whereas the remainder will be straightened to cross open water portions of the pond. Sections of the existing salt pond levee remaining within the bypass channel alignment would be removed following the new levee construction.

It is proposed to position the ultimate top of the new Reach 1A levee at Elevation 13. This will provide about 4 feet of freeboard above the 1 percent design flood level. It also is proposed that a levee crest width of 30 feet be provided to serve as an access road for maintenance equipment. Improvements to the existing levee along the south side of Newby Island also are planned. However, that construction will be handled by the Newby Island Improvement Company and the levee design considerations are beyond the scope of this study.

SITE HISTORY

Based on a discussion with Mr. Raymond Thinggaard of Leslie Salt Company, it is understood that the levees surrounding the salt evaporation pond were begun in the early 1930's using dredging techniques. These levees have been periodically raised and maintained over the years by dredging from a "borrow ditch" along the inboard toe of the levee. It is understood that the levees were most recently topped-up during the fall of 1982 through the winter of 1983. According to Mr. Thinggaard, soils were dredged from within the borrow ditch until "hardpan" was encountered by the clamshell bucket. An aerial view of the evaporation pond showing the configuration of the borrow ditch as well as the pattern of sloughs which were tributary to Coyote Slough is presented on Figure 2, a 1971 aerial photo taken when the pond was nearly dry. Based upon a review of "Farm Map Number Two" of Santa Clara County surveyed in the mid-1800's (New Historical Atlas of Santa Clara County, 1876), it appears that the easterly most slough within the salt pond area actually extended northeast through Newby Island to connect with Coyote Creek. Surface features shown on a 1928 aerial photo in the Leslie Salt Company archives suggest that the historical extension of the slough probably followed the alignment shown on Figure 2.

TOPOGRAPHIC AND HYDROGRAPHIC FEATURES

Topographic information provided by the District from an aerial survey in June 1983, indicates that the top of the existing salt pond levee generally is positioned between about Elevation 9 to 10-1/2 (U.S.G.S. mean sea level datum updated in 1976 by the District). The irregular dredged surface of the levee generally slopes downward from the crest to the pond water level (approximate Elevation 3-1/2±) at inclinations of about 3:1 (horizontal to vertical) to 4:1. Hydrographic data indicates that the pond bottom generally is positioned between about Elevation 0 and 2; although the existing borrow ditch and old sloughs have bottoms which extend as deep as Elevation -8 in the Reach 1A area.

SUBSURFACE CONDITIONS

Exploration and Testing

Borings 1 through 7 were drilled within the salt evaporation pond area to explore the subsurface conditions below the pond water surface. Borings 2A, 3A and 5A were drilled from the top of the salt pond levee at locations adjacent to borings in the pond to permit comparisons of the soils in the respective areas. A combination of laboratory testing of samples recovered from the borings and insitu field vane shear testing then was utilized to estimate the engineering properties of the materials which were encountered. These procedures are discussed in detail in Appendix A.

Salt Pond Area

The borings made within the salt pond area generally encountered a thin (1± inch) salt crust covering the pond bottom. Beneath the salt crust, the stratigraphy within the depth explored can be generalized as consisting of very soft, highly plastic silty clays locally known as "young Bay Mud", overlying stiff to very stiff older silty clay. In general, the Bay Mud layer was found to be less than 10 feet in thickness along most of the Reach 1A levee alignment. The exceptions are at Boring 1, where about 7 feet of Bay Mud were found to overlie loose to medium dense, black silty sand with interbeds of Bay Mud and shells extending to more than 19 feet; and in Boring 7, where 17 feet of Bay Mud were encountered. It is likely that the increased thickness of Bay Mud (or Bay Mud and sand) at these boring locations can be attributed to their proximity to historic sloughs or channels shown on Figure 2. An idealized soil profile (Sections A-A') depicting the subsurface conditions along Reach 1A is presented as Figure 3a. The profile has been constructed by direct interpolation between Borings 1 through 7.

Typically the Bay Mud encountered in the salt pond area exhibits dry densities of about 60 pcf with moisture contents in the range of 60 to 70 percent. Its consistency is very soft with undrained shear strengths estimated by both laboratory and insitu field vane testing to range from a low of about 50 psf to a high of more than 400 psf; although a compilation of the test data suggests that the strength profile within the Bay Mud probably increases from about 100 psf at the surface to about 200 psf at a depth of 10 feet. Remolded shear strengths estimated by the field vane (Table A-3) were found to be about 1/2 to 1/3 of the corresponding peak values indicating that the Bay Mud deposit is of low to moderate sensitivity. The results of a consolidation test on Bay Mud performed for this investigation and for other studies in the general vicinity indicate that the preconsolidation pressure is near or slightly above the existing overburden pressure and the Bay Mud is probably in a normally or slightly overconsolidated condition.

Below the Bay Mud layer and extending to the limits of Borings 1 through 7, the older clays were found to exhibit dry densities generally in excess of 100 pcf and moisture contents between 20 and 30 percent. These clays are considered to be relatively strong and of relatively low compressibility.

Existing Salt Pond Levee

The existing salt pond levee is comprised of dredged Bay Mud. Generally the upper 2 to 3 feet consist of relatively stiff blocks of recently dredged material which exhibit wide shrinkage cracks due to dehydration. Below this top layer, the dredged material generally was found to be medium in consistency. The dredged material is in turn underlain by natural Bay Mud which extends to depths comparable to those encountered in the adjacent borings drilled in the pond area. The natural Bay Mud is soft to medium in consistency suggesting that strength gain has occurred due to consolidation of the layer under the load imposed by the dredged fill. The results of field vane tests of the levee fill and underlying natural Bay Mud are presented on the Logs of Borings 2A, 3A and 5A (Figures A-3, A-5 and A-8) and on Table A-3.

DISCUSSION AND RECOMMENDATIONS

The low strength and high compressibility of the Bay Mud underlying the salt evaporation pond area are the principal geotechnical factors influencing the design and construction of the proposed Reach 1A levee. Historically levees in the former marshland areas of the bay have been constructed by dredging as was done for the existing levee surrounding the salt pond. Although this method offers the advantages of low initial cost and fill materials which have low unit weight, it can require many years to achieve the desired levee height because of the weak, compressible properties of the dredged fill material. Since the District considers that levee construction by dredging would have longterm cost and maintenance implications, it was excluded as a viable alternative. For this reason, levee construction alternatives utilizing imported fill materials and/or on-site soils borrowed from other areas along the project alignment were analyzed.

Based upon the results of slope stability studies performed for this investigation and previous experience gained at other sites with similar subsurface conditions, it is expected that the Bay Mud probably would be capable of supporting an imported fill levee with 5:1 (horizontal to vertical) side slopes to a maximum height of about 9 feet, providing that a layer of geotextile stabilization fabric was placed across the pond bottom to help distribute the new fill load. Assuming the typical pond bottom to be positioned near Elevation 0, this would result in an initial levee profile of about Elevation 9. Based on experience, levee fill sections exceeding this height might cause shear failure in the Bay Mud resulting in the development of extensive mud waves.

It is our opinion that there are three alternative methods which could be used to construct the proposed Reach 1A levee through the salt pond area. The alternative methods consist of: (1) removal of the existing Bay Mud by dredging a channel beneath the proposed levee alignment prior to constructing the levee fill section; (2) purposely inducing shear failure and mud waves to displace the Bay Mud; and (3) floating the new levee embankment over the Bay Mud using a system of lateral berms and polymer grid reinforcement to enhance the resistance to shear failure.

Of these three methods, the dredging alternative would be expected to be the most costly but would offer the least construction risk and uncertainties. It would have the advantage of virtually eliminating the risk of shear failure in the Bay Mud, as well as the potential for future levee settlement due to Bay Mud consolidation. The displacement method would also offer similar advantages at a somewhat lower cost, but the extent and uniformity of the Bay Mud removal would be more difficult to control than by dredging.

Bay Mud not totally displaced could be trapped which might result in some irregular levee settlement. The floating embankment approach would probably be the least costly since the lowest initial volume of levee fill would be required. However, this procedure presents the greatest risk and uncertainties during construction. Periodic topping of the levee would be needed after construction as the underlying Bay Mud consolidates. Furthermore, and more important, there is a risk that shear failure and the development of a mud wave could occur in the Bay Mud as the levee fill is being placed. In the event that a mud wave was to be initiated, it could be difficult (if not impossible) to continue with the floating technique. In the event that the costs of the displacement and floating embankment alternatives are similar, the displacement method would be preferable because of its lower risk. Each of the alternative methods are described in detail in the following paragraphs.

Dredging Alternative

Along those sections of the alignment which will adjoin the existing salt pond levee, it appears that the new levee will fall within the limits of the existing borrow ditch. Based on comparisons of the Bay Mud layer depths encountered in the borings with hydrographic and cross section data provided by the District, it appears that much of the Bay Mud already has been removed in these areas. Dredging of the remaining Bay Mud would permit the construction of the major part of the new levee section directly on the stronger underlying clays, thereby minimizing future levee settlement, as well as the potential for shear failure of the Bay Mud. Similarly, dredging could be used to remove the Bay Mud from beneath the sections of the alignment where the new levee will cross open water areas of the salt pond. It is estimated that the dredging depths in these areas probably would be on the order of 10 feet; although a greater dredging depth should be expected near old sloughs. If dredging is to be used, it is recommended that the open water areas be probed before dredging begins, to provide a better estimate of the depth of the Bay Mud.

For this alternative, it is recommended that the Bay Mud be dredged to the approximate limits shown on Figure 4. Details regarding levee construction at locations adjoining the existing salt pond levee and in the open water areas are shown on that figure. If dredge spoil is to be discarded in areas adjoining the levee alignment, it should be done so beyond the limits of a 6:1 (horizontal to vertical) plane projected from the base of the dredge cut to the existing pond bottom surface. The intent of this recommendation is to prevent the development of a mud wave into the excavated channel due to surcharge loading from the spoil.

It is expected that underwater cut slopes in the Bay Mud materials should stand temporarily at inclinations of 1:1 for the anticipated excavation depths. During dredging, it is recommended that the District's representative probe the bottom area using a sounding rod to confirm that the soft muds have been removed in their entirety.

After the dredging operation has been completed, the new levee section can be constructed. Fill placed underwater should consist of a granular "bank run" material meeting the general requirements outlined in "General Recommendations" (Section 5). The underwater fill should be placed in such a manner as to prevent segregation of the fill as well as the entrapment of any soft materials which might remain on the excavation bottom. For this reason, it is recommended that the Geotechnical Engineer review the Contractor's proposed method of underwater fill placement prior to construction.

When the level of the granular fill has been raised above the pond water surface and a stable base has been achieved, the above water portion of the levee should be completed using cohesive embankment fill materials meeting the requirements outlined in the attached specifications. It is estimated that it might be necessary to place the granular fill to a height of about 3 feet above the pond water surface before truck and compaction equipment traffic could be sustained. Placement and compaction of the above water portion of the levee fill, including details regarding the preparation of the existing salt pond levee to receive new fill, are discussed under "General Recommendations". It is recommended that the slopes of the completed levee section not exceed 2:1 (horizontal to vertical).

It is estimated that the post construction levee settlement for this method might be of the order of 3 to 6 inches in areas where nearly complete Bay Mud removal is accomplished. Where the new levee will adjoin the salt pond levee, much of the existing dredged fill and underlying natural Bay Mud will remain. It is estimated that settlements along this adjoining edge of the new levee might range from about 12 to 18 inches, depending on the thickness of the compressible materials and the consolidation which already appears to have occurred as a result of the existing dredged levee fill placement.

Displacement Alternative

It is our opinion that construction of the proposed Reach 1A levee through the salt pond area also could be accomplished by displacing the very soft Bay Mud during the placement of the levee fill. Based on experience, it is believed that the majority of the Bay Mud layer could be displaced providing that the fill is advanced in a systematic manner.

It is recommended that this be accomplished by end dumping a displacement fill of imported bank run material directly on the Bay Mud starting at one end of the levee alignment (preferably the east end) and advancing the fill to the other end. Bank run material should be coarse, angular rock material meeting the requirements as described in "General Recommendations" (Section 5). It is estimated that about a 6 to 8 foot thickness of end dumped fill placed at the natural angle of repose of the material might be required to initiate shear failure and the formation of the mud wave. In order to achieve the maximum penetration of the fill and displacement of the Bay Mud, as well as minimize the potential for entrapping Bay Mud, it is recommended that the displacement fill be advanced using a wedge-shaped nose with the sides of the wedge at about 30 to 45 degrees from the centerline. It is expected that further efforts to advance or push the mud wave in a preferred direction probably would not be practical.

As shear failures occur under the weight of the advancing fill, additional material should be placed in the failed areas to restore them to the displacement fill grade. In order to keep the mud wave moving it is likely that the top of the advancing nose of the displacement fill will have to be kept about 3 to 5 feet above the crest of the mud wave.

In the event that this results in the top of the advancing nose being less than Elevation 13, it is recommended that additional fill be placed to that minimum height to confirm that the majority of the Bay Mud has been displaced and the underlying materials will be capable of supporting the design levee section. This uncompacted fill then should be removed or dozed ahead with the advancing nose such that the remaining working pad surface would be positioned about 3 feet above the pond water surface. The limits of the displacement fill also should extend at least 10 feet beyond the toe of the new levee as shown on Figure 5, the intent being to displace as much Bay Mud as possible from beneath the edge of the design levee section.

At the transition between the open water sections of the levee and where the levee will adjoin the existing salt pond levee, it is expected that a portion of the advancing mud wave would be trapped between the nose of the working pad and the existing levee. For this reason, it is recommended that the portion of the mud wave to the existing levee side of the new embankment centerline be excavated as the working pad approaches to within about 100 feet of the existing levee. The final need for and extent of the mud wave excavation should be determined by the Geotechnical Engineer at the time of construction. It is expected that the excavation of the Bay Mud in these transition areas could be accomplished by dragline equipment operated from the top of the salt pond levee.

After the working pad has been completed, cohesive fill can be placed and compacted in lifts as described in "General Recommendations" to bring the levee up to the design crest elevation. It is recommended that the levee slopes be inclined no steeper than 2:1 (horizontal to vertical). Details regarding the recommended procedures described above are shown schematically on Figure 5.

It is estimated that the post construction settlement for this levee construction method might be about 6 inches. This is based on the assumption that about 75 percent of the Bay Mud would be displaced. If thicker deposits of Bay Mud are trapped, larger settlements would be expected. Where the new levee will adjoin the existing salt pond levee, it is estimated that settlements on the order of 12 to 18 inches might be expected.

The lateral extent of mud waves developed during the displacement fill construction is difficult to estimate because of the variable thickness and strength characteristics of the Bay Mud layer. However, as a rough approximation, it is estimated that they might range from about 150 to 200 feet from the levee centerline.

If nearly complete displacement of the Bay Mud is achieved, it is anticipated that the mud wave could be removed to the original pond bottom surface within 6 months or less following completion of the levee construction. Should only partial displacement be achieved, weak zones would result beneath the completed levee. In this case, the mud wave would help to counter balance the driving forces of the levee fill and some strength gain in the underlying materials would be needed before the mud wave could be removed. It is conceivable that a period of 1 to 2 years might be required to attain a strength gain sufficient to allow the removal of the mud wave. It is recommended that periodic estimates of the strength gain be made by means of insitu testing of the Bay Mud to help assess when mud wave removal would be advisable.

Floating Embankment Alternative (Elevation 13)

It is our opinion that a third alternative method could be used to construct the proposed Reach 1A levee to the desired top Elevation 13. This method would incorporate a layer of polymer grid reinforcement material (Tensar Geogrid SS2 or equivalent) at the interface between the Bay Mud and levee fill section in conjunction with lateral berms extending beyond the levee toe. The intent of using the grid reinforcement would be to help more evenly distribute the loads imposed by the levee fill, as well as to develop resistance to shear failure of the levee foundation through the tensile strength of the material. The lateral berms would help to counter-balance the driving forces of the main levee section.

This alternative method would require that the grid reinforcement be placed to cover the pond bottom for the full width and length of the levee alignment. Adjacent rolls of the grid material would be joined by tying or overlapping them in accordance with the manufacturer's recommendations to provide continuity of reinforcement. After the grid material is in place, imported granular "bank run" material then could be end dumped and spread with light weight track-mounted equipment to form a working pad extending above the pond water level. The bank run material should meet the requirements outlined in "General Recommendations (Section 5). When the working pad has been completed, cohesive embankment fill then could be placed and compacted to form the final design levee section.

Additional layers of grid reinforcement could be used with this system to help increase the tensile strength of the levee and the factor of safety against levee instability. However, it is doubtful that an analysis of the costs for these additional layers versus the strength increase would justify their use.

The pond bottom profile is variable along the proposed levee alignment due to natural interruptions by sloughs and the man-made borrow ditch excavated during dredging operations. In addition, the Bay Mud layer thickness also is variable. Selection of the thickness and extent of berms to counterbalance the driving forces associated with the varying levee fill sections and Bay Mud thickness is partly a function of these variables. Recommendations for floating embankment construction along sections which have differing design and construction considerations are discussed in the following paragraphs.

Adjoining the Salt Pond Levee - Based upon a comparison of the hydrographic data and cross sections prepared by the District with the subsurface conditions encountered in the borings, it appears that some portions of the borrow ditch probably contain only a thin layer of Bay Mud; whereas at other locations the thickness might be on the order of 4 to 5 feet. Conceivably, some portions of the new levee could be constructed without a lateral berm. However, extensive probing would be needed to assess the limits of the areas where near total Bay Mud removal has been achieved by the previous dredging operations. In lieu of attempting to select the berm dimensions to correspond with these varying conditions, it is recommended that a single section be used. The recommended levee and berm dimensions are presented on Figure 6. Those dimensions have been selected based upon the assumption that a Bay Mud thickness of about 4 to 5 feet would underlie the new levee area.

Open Water Areas - Where the proposed levee will traverse open water portions of the pond, it is expected that most of the pond bottom will be relatively uniform and positioned between about Elevation 0 and 2. For this general condition, it is recommended that the levee and lateral berm dimensions shown on Figure 6 be used for design. A berm thickness of 5 feet has been recommended based upon levee stability considerations. However, it is expected that a minimum height of fill above the pond water surface of 3 feet probably will be needed in areas which will be subject to repeated truck or compaction equipment traffic.

Slough Areas - Where the proposed new levee alignment crosses the old sloughs shown on Figure 2, fill thicknesses of more than 20 feet are expected to be needed to provide the design top of levee at Elevation 13. This conclusion is based on the available hydrographic data which position the bottoms of the deepest sloughs at about Elevation -8. Boring 7, which was drilled near the slough closest to the east side of the pond, encountered Bay Mud to about Elevation -15. Therefore, it is expected that about 7 feet of Bay Mud probably underlie the adjoining slough. Lesser thicknesses of Bay Mud are expected to underlie other sloughs along the new levee alignment.

Based upon the results of stability calculations for increased thicknesses of fill overlying Bay Mud, it is probable that shear failure and Bay Mud displacement would occur at some of the deeper slough locations even if broad lateral berms and grid reinforcement are to be provided. Since shear failure appears likely, the levee construction sequence should be planned accordingly to minimize the consequences.

When the working pad has been advanced to the edge of the existing sloughs which extend deeper than Elevation -4, it is recommended that the end dumping of bank run material be continued without the use of the grid reinforcement. The fill should be advanced across the sloughs using a wedge-shaped nose as previously discussed for the displacement alternative. It is expected that this will induce mud waves laterally along the direction of the slough axis. The working pad should be advanced across the sloughs in this manner with the fill being placed until an equilibrium condition has been achieved and the pad surface is raised to about 3 feet above the pond water surface. The final top width of the working pad surface should match that recommended for the lateral berms shown on Figure 6. When the opposite side of each slough has been reached, placement of the grid reinforcement should be resumed prior to advancing the working pad further along the alignment.

As cohesive embankment fill is placed and compacted above the working pad level, it is expected that additional Bay Mud probably will be displaced by squeezing from beneath the pad resulting in its settlement. Since it also would be preferred to displace this Bay Mud away from the levee along the direction of the slough axis, it is recommended that the fill be raised simultaneously on each side of the slough. As further displacement and working pad settlement occur, the pad grade of the lateral berms should be restored by end dumping and dozing additional material in place.

It is anticipated that it might be necessary to switch from floating to the displacement technique at other locations along the alignment, such as where the levee would parallel existing sloughs. For this reason, it should be expected that construction by this technique could be difficult to administer.

Transition Areas - As the open water portions of the levee working pad are advanced toward the existing salt pond levee, it is expected that Bay Mud could be displaced into the borrow ditch. In order to minimize the potential for entrapping this material between the new and existing levees, it is recommended that the displaced mud be excavated. It is expected that this could be accomplished using dragline equipment operated from the top of the salt pond levee. The excavation of the displaced mud should extend to the original depth of the borrow ditch.

As an alternative to excavating the displaced mud, the grid reinforcement and bank run fill could be placed to raise the borrow ditch bottom elevation to match that of the surrounding pond bottom prior to advancing the working pad into these areas. This would be expected to minimize the potential for the displacement of Bay Mud.

Floating Embankment Alternative (stage construction)

It is understood that stage construction might be considered for the Reach 1A levee if an initial crest of Elevation 9 can be provided. A system of lateral berms and grid reinforcement would still appear to be advisable for this first stage of floating embankment construction; however, it is estimated that the berms could be somewhat narrower. Recommended berm dimensions for the initial levee construction to Elevation 9 are shown on Figure 7.

Settlement

The settlement of levees constructed by the floating embankment method will be a function of the total fill thickness and the thickness of the compressible Bay Mud layer. A graphical presentation of estimated total settlement versus Bay Mud thickness for a range of fill thicknesses is presented on Figure 8.

Settlement of a levee floated on the Bay Mud will occur as the Bay Mud consolidates under the applied load. Since the Bay Mud is underlain by stiff, relatively impermeable clay along most of the alignment, it is expected that drainage of the consolidating layer will take place at the top into the granular fill. For this condition of single drainage, it is estimated that about 50 percent of the settlement will occur in 2 years and 90 percent in about 8 years assuming an average Bay Mud thickness of 10 feet. Where the levee would be underlain by 17 feet of Bay Mud, the corresponding times for 50 and 90 percent settlement to occur would be expected to increase to about 6 and 25 years respectively.

It should be recognized that these estimated time rates of settlement are based on the assumption of single drainage. In actuality, it is likely that other drainage paths exist within the Bay Mud which will allow for a more rapid rate of consolidation. If it is desired to further accelerate the consolidation, vertical drains could be installed to improve the drainage characteristics of the Bay Mud layer. Prefabricated, band-shaped "wick drains" installed by mandrel equipped machinery would be expected to be effective. Wick spacings on the order of 3 to 4 feet on centers probably would be required. However, the use of wick drains would be expected to result in a substantial increase in construction costs.

Future Fill Placement

As settlement of the levee occurs, additional fill will be required to maintain the design crest elevation or, in the case of the stage construction of a floated embankment, to also increase the height. The higher strength values estimated by vane shear testing in the natural Bay Mud beneath the existing salt pond levee, as well as experience at other Bay Mud sites, indicate that strength gain in the Bay Mud layer will occur as excess pore pressure dissipates and the layer consolidates under the weight of the levee fill. It is expected that the short term strength gain will be capable of sustaining thin fills placed to restore the design top of levee elevation as the settlement occurs. However, it is likely that 2 years or more would be required before the levee could be raised from its initial stage of Elevation 9 to the design height.

Because of the uncertainty involved in predicting the rate of strength gain, it is recommended that a program of instrumentation consisting of settlement plates and piezometers plus insitu vane shear tests be provided to monitor the behavior of the Bay Mud during and after construction. It is suggested that monitoring points be established at 500 foot intervals along the levee alignment. During construction, the Geotechnical Engineer should be consulted to develop details regarding the system of instrumentation and frequency of monitoring.

Test Section

As previously discussed, there is a risk of Bay Mud shear failure and the development of mud waves for the floating embankment alternative. If this levee construction alternative is to be selected, it is recommended that consideration be given to constructing a test section under a separate construction contract in advance of the entire Reach 1A levee. In this way, information regarding the test section performance could be used in the preparation of the final plans and specifications. The test section should be constructed in the open water portion of the alignment (preferably beginning at the east end) and should be at least 400 feet in length.

If the displacement alternative is to be used, it is recommended that a test section of similar length also be considered. In this way, the height of fill needed to initiate and advance the mud displacement could be determined. The test section should be designed in accordance with the recommendations presented above for the displacement alternative. It is recommended that the Geotechnical Engineer be retained during the test section construction. By doing this, the Geotechnical Engineer could review the effectiveness of the test section construction operations and recommend modifications for incorporation into the final levee construction contract. Exploratory borings drilled through the completed test section should be made to assess the extent of mud displacement.

Underseepage

Construction of the proposed levee across the salt pond by any of the alternative methods discussed in this report would involve the use of a granular bank run material end dumped to raise a working pad above the pond water level. Two samples of bank run materials from local quarries were compacted and tested in the laboratory to estimate their respective permeabilities. Each was found to have a permeability (k) of about 1×10^{-4} cm/sec (Table 2).

Although a permeability on this order is moderately low, it is likely that some segregation and stratification would occur as the material is end dumped underwater rather than being uniformly spread and compacted. Consequently, it is expected that the effective permeability of the working pad probably would be higher.

Based on the assumption that the permeability of the working pad might be as much as an order of magnitude larger than that of the compacted samples, it is probable that the underseepage through the open water sections of the levee (approximately one-half of the Reach 1A alignment) could be several thousand gallons per day. In order to minimize the seepage loss, it is recommended that a blanket of Bay Mud not less than 18 inches thick be provided against the bank run portion of the levee (including the lateral berm).

Other methods of seepage reduction, such as grouting or cutoff walls might be technically feasible. However, it is expected that they would be considerably more costly than the recommended Bay Mud cover.

Wave Action

Wind developed waves will cause erosion of the levee near the water surface. The need for erosion protection should be considered during design. A coating of uncompacted Bay Mud placed on the sides of the levee might be considered as a possible means of erosion protection.

PROPOSED IMPROVEMENTS

The proposed bypass channel along the south side of Newby Island will be formed by constructing a system of new flood plain levees as shown in plan view on Figure 1b. This will result in a bypass channel width ranging from about 800 to 1,500 feet.

The levee along the south side of the alignment will adjoin, as well as pass through, portions of the sludge lagoon levees of the San Jose-Santa Clara Water Pollution Control Plant. It is planned that this section of new levee will conform to the tops of the existing sludge lagoon levees resulting in a levee surface profile ranging from about Elevation 13 to 16 from the west to the east end of the alignment, respectively. A final crest width of 30 feet is planned for this section of levee. In addition to the new south levee construction, it is planned to relocate the City of San Jose Bomb Disposal Facilities to a site at the west end of the levee, as shown on Figure 1b.

The levees proposed for the north and east sides of the Reach 1B area also are expected to have similar design profiles as the levee along the south side of the channel. However, these levees will have narrower crest widths of 18 feet. Because of right-of-way constraints along the north side of Coyote Creek near the existing Dixon Landing Road bridge over the creek, it is proposed to construct about 600 feet of reinforced concrete cantilever retaining wall to serve as a floodwall. It is planned to position the top of the floodwall at Elevation 13. The approximate location of the proposed floodwall is shown on Figure 1b.

In addition to the levee and floodwall improvements described above, it also is planned to regrade the upstream portion of the flood plain area to a finished grade of Elevation 7.

The present meandering course of Coyote Creek also will be straightened to provide a new low flow channel. The abandoned section of the existing creek then should be backfilled to match the general flood plain grade at Elevation 7. Details regarding the channel profile were not available at the time of this writing.

SITE AND GEOTECHNICAL FEATURES

The Reach 1B portion of the project alignment has been modified extensively over the last century. A study by the United States Geologic Survey (1971) using Coast and Geodetic Survey topographic maps from the mid-1800's indicates that the historic margin of marshland probably extended somewhat beyond Station 90 of the proposed bypass channel alignment. It is likely that the earliest modifications were made around the turn of the century and consisted of the extension of Dixon Landing Road and the construction of a levee along Coyote Slough to reclaim portions of Newby Island. Dredged levees creating the salt evaporation pond in the mid-1930's and subsequent levee construction now limit the tidal influence of the San Francisco Bay to the area bordering the San Jose Scavenger Company property as shown on Figure 1b.

More recent private and local government development also has modified the area. That development included the construction of the Milpitas Sanitation District facilities in about 1965 and the sludge lagoon additions to the San Jose-Santa Clara Water Pollution Control Plants in 1972. These developments can be seen on Figure 1b.

During the late summer and fall of 1983, the Santa Clara Valley Water District implemented the construction of "Interim Flood Control Improvements" along Coyote Creek. Those improvements were described in the District's "Coyote Creek Engineer's Report," dated June, 1983. A copy of that report is attached for convenience of reference as Appendix B.

The following paragraphs describe the different surface and subsurface conditions which exist within the Reach 1B area.

Surface Features

Channel Area - Topographic information provided by the District (June, 1983) indicates that the ground surface in the Reach 1B portion of the proposed bypass channel area slopes gradually upward from about Elevation 1 near the salt pond levee at the west end of the alignment to about Elevation 10 in the vicinity of the Milpitas Sanitary District facility. Man-made and natural drainage features are present within the Reach 1B area; however, details regarding the respective depths of these features were not available at the time of this writing.

South Levee Alignment - The south side of the proposed bypass channel alignment is flanked by the sludge lagoons of the San Jose-Santa Clara Pollution Control Plant. As-built drawings prepared by Consoer, Townsend & Associates dated May, 1971, indicate that the lagoons were constructed by excavating from within the lagoon bottom areas and placing the borrowed material as compacted levee fill. The drawings indicate the lagoons were designed with a 12 foot depth, 2:1 (horizontal to vertical) side slopes and a 15 foot crest width. Topographic information (June, 1983) indicates that the tops of the levees near the west end of the alignment are generally positioned between about Elevation 13 and 13 1/2; whereas the top of the levee near the east end of the Reach 1B alignment (across from the Milpitas Sanitation District facility) is positioned between about Elevation 16 to 16 1/2. Therefore, it is expected that the lagoon bottoms are positioned at about Elevation 1 and 4 in the west and east lagoons, respectively. Trapezoidal shaped ditches were excavated along the toes of the sludge lagoons to provide for the drainage of storm water runoff. These toe drains were designed with 2:1 (horizontal to vertical) side slopes, bottom depths of about 5 to 6 feet below the original ground surface, and bottom widths of 10 feet and 5 feet for the west and east levees, respectively. The drains now are generally much shallower than designed and appear to contain soft sediments eroded from the surrounding areas.

During the summer and fall of 1983, a short section of levee with its crest at Elevation 10 was constructed at Site #2 (Appendix B) across the drainage channel between the salt pond levee and the northwest corner of the sludge lagoon levee as part of the District's "Interim Flood Control Improvements".

North and East Levee Alignment - During site reconnaissance for this study, a narrow dredged levee was observed to be set back about 5 to 10 feet from the edge of the San Jose Scavenger Company property where it is bounded by the existing tidal marsh area (approximate Station 70 to 80). The dredged levee has a top width of about 3 feet, side slopes of about 1-1/2:1 (horizontal to vertical) and stands about 4 feet above the general site grade along the edge of the property. Below the dredged levee, the edge of the property slopes toward the tidal marsh at an inclination of about 1:1. Available topographic information positions the top of the dredged levee at about Elevation 10 to 12 in this area and the surface of the adjacent tidal marsh at about Elevation 4-1/2. The tidal marsh area presently supports a dense growth of bulrush.

East of the tidal marsh area, the District recently completed the construction of a section of levee at Site #3 (Appendix B) as part of their "Interim Flood Control Improvements". This interim levee parallels the drainage channel along the extension of Dixon Landing Road, extends around the south side of the Collier Briquet plant and then northeast along the plant property line to the west abutment of the Dixon Landing Road bridge over Coyote Creek (approximate Station 80 of the bypass channel to Coyote Creek Station 440). The Engineer's Report called for the crest of this section of levee to be positioned at Elevation 10. During site reconnaissance, the side slopes of this interim levee were estimated to be inclined at about 1-1/2:1 (horizontal to vertical).

To the east of the interim levee, in the area along which the new levee is planned to conform to the curving alignment of the existing channel, the ground surface is somewhat irregular and ranges from about Elevation 4-1/2 to 8-1/2. The creek bank along portions of this area has been undercut to a near vertical inclination and is covered by a dense growth of bulrush and other grasses.

On the north side of Coyote Creek, Dixon Landing Road is positioned several feet above the natural grade to the north by means of an old embankment fill. Concrete rubble is evident along the banks of the roadway embankment. The road grade along the proposed floodwall alignment is positioned at about Elevation 10-1/2 to 11.

Extending south across lower Penitencia Creek, the ground surface along the east levee alignment gradually rises from about Elevation 5 to more than Elevation 9 at the north corner of the Milpitas Sanitary District facility. The ground surface within the facility property ranges from about Elevation 7-1/2 in the existing sludge and settling pond areas to almost Elevation 14 on the tops of the levees which surround them. It is understood that the bottoms of the ponds were originally designed to extend to about Elevation 7, but could be deeper. As shown on Figure 1b, the proposed levee alignment will traverse the existing ponds and areas now occupied by existing treatment plant structures.

Subsurface Conditions

Exploration and Existing Data - Borings 8 through 17, P-1 and P-2, and Exploratory Pits 1 and 8 through 10 were made within the Reach 1B portion of the project alignment to supplement existing available subsurface data from previous studies. The locations of these explorations and those from previous studies in the Reach 1B area are shown on Figure 1b. Descriptions of the exploratory procedures used for this study are presented in Appendix A.

Extent of Bay Mud - In general, it was found that much of the Reach 1B area is underlain by a sequence of very soft to soft Bay Mud overlying stiff, older clays similar to the conditions which were encountered in the salt pond area. Borings made by Dames & Moore (1971) in the low lying former marshland area just east of the salt pond encountered Bay Mud at the ground surface. However, moving upstream, the Bay Mud (where present) has been covered by younger alluvial deposits which consist primarily of low plasticity clayey silts to non-plastic silts and fine sands. Based upon interpolations between the exploratory borings, it is estimated that the Bay Mud might extend inland to the approximate limits shown on Figure 1b.

South Levee Alignment - In order to generalize the subsurface conditions encountered along the south levee alignment, an idealized soil profile (Section B-B') has been prepared and is presented as Figure 3b. In the westerly part of the reach, closest to the salt pond, the detonation site and a portion of new levee alignment are expected to be underlain by Bay Mud which probably decreases in thickness from about 7 feet near the salt pond to nil at about Station 80.

Although the previous boring data (DM-38, 39 and 40) indicated that Bay Mud was likely to be found beneath the existing sludge lagoon levee, Boring 8 encountered only silty clay levee fill over very stiff native clays. Since the depth of levee fill extends several feet below the original site grade in that area, it is believed that the Bay Mud layer probably was excavated during the sludge lagoon construction. Sampler penetration resistance values coupled with the results of laboratory strength and density tests indicate that the existing sludge lagoon levee fill is in a moderately to well compacted condition with consistencies comparable to that of stiff to hard clays.

Although no exploration was made in the short section of interim levee between the salt pond levee and sludge lagoon, visual examination suggests that these materials probably were bulldozed in place with little compactive effort.

In the central part of the reach, moderately to well compacted silty clay levee fill also was found to overlie predominantly stiff silty clay alluvium. Shallow surficial layers of sandy silt to fine silty sand were recorded in the previous borings (DM-36 and 37); however, it appears that these deposits also were removed in preparation for the sludge lagoon levee construction. Just east of the lagoons, shallow trash fill was encountered in Exploratory Pit 9. It is suspected that this is an isolated condition; however, other such pockets might exist in the open area between the sludge lagoons.

Further east, the shallow alluvium in the open area between the sludge lagoons was found to grade from loose sandy silt to medium dense silty sand. Medium dense native silty sand also was encountered below the moderately to well compacted levee fill in Boring 17. These cohesionless alluvial deposits appear to grade somewhat coarser moving toward Coyote Creek. They are, in turn, underlain by stiff to very stiff silty clays similar to those encountered beneath the entire south levee alignment.

North and East Levee Alignment - An idealized soil profile (Section C-C') also has been constructed by direct interpolation between borings drilled along the proposed north and east levee alignment and is presented as Figure 3c. Unlike the south levee alignment, it appears that a major portion of the north and east levee areas are underlain by Bay Mud.

As previously discussed, Bay Mud is expected to be exposed at the ground surface in the low lying former marshland area just east of the salt pond. Although explorations were not made in the tidal marsh area, it is suspected that more than 3 feet of very soft tidal deposits (probably clays and silts) and organic material probably overlie the Bay Mud. This opinion is based on the elevation difference between tidal marsh (Elevation 4-1/2±) and the adjacent, former marshland (Elevation 1±).

Further upstream, the Bay Mud is typically overlain by several feet of young alluvium. The young alluvium consists of loose to medium dense sandy silts to silty sands and low plasticity clayey silts to silty clays which are generally soft to medium in consistency. These deposits are often highly gradational and interbedded. The young alluvial deposits also were encountered in areas where the Bay Mud is absent. At these locations, the underlying deposits were found to consist primarily of stronger, more plastic silty clays which grade to very stiff clay with increasing depth.

In addition to the native soil conditions described above, portions of the alignment were found to be covered by varying thickness of fill. As previously discussed, a narrow dredged levee is located along the edge of the San

Jose Scavenger Company property. Boring 9, which was drilled on the landward side of the dredged levee, encountered about 13 feet of trash and soil fill. The trash and soil fill was in turn underlain by stiff to very stiff native clay layers similar to those encountered elsewhere along the alignment.

Based upon visual examination, as well as a shallow exploratory pit (EP-10), it appears that the soils used for construction of the District's interim levee are primarily cohesive and of medium to high plasticity. At the time of site reconnaissance for this study, the levee fill appeared to have been bulldozed in place with little compactive effort.

Boring 13 was drilled to the east of the Collier Briquet plant property line. The upper 3 feet of soils were found to consist of fill with fragments of nut shells or fruit pits. It is suspected that much of the irregular ground surface in the area bounded by Coyote Creek and the plant might be attributed to similar shallow fills.

As discussed in "Surface Features", concrete rubble is scattered along the edge of Dixon Landing Road in the area which has been proposed for the construction of a new floodwall. Although no explorations were made in the roadway area, it is expected that the existing roadway fill section probably consists of a mixture of soil and rubble fill materials.

Shallow surface fills also are evident along lower Penitencia Creek and further south within the Milpitas Sanitation District facility. It is expected that these fills are on the order of 2 to 5 feet thick.

Groundwater - Free groundwater was recorded in explorations made both during this study and previous studies in the Reach 1B area. Near the west end of the alignment, the water levels generally have been found to range from about Elevation -1 to -2. Further to the east, the water levels were recorded between about Elevation 1 and 4. It appears that water level in the easterly portion of Reach 1B generally corresponds with the water level in Coyote Creek.

DISCUSSION AND RECOMMENDATIONS

South Levee Alignment

The subsurface conditions along the proposed south levee alignment range from very soft Bay Mud in the westerly part of the reach to low and moderate strength alluvium along the remainder. Explorations indicate that the existing sludge lagoon levees are composed of moderately to well compacted fills which are predominantly native clays and silts borrowed from within the lagoon areas. These conditions form the basis for the recommendations which follow.

Salt Pond to Station 80 - It is estimated that very soft Bay Mud underlies the proposed detonation site to a depth of about 7 feet and gradually becomes thinner moving east to about Station 80. For this portion of the alignment, it is recommended that the new levee and detonation site fill either be floated over the Bay Mud in the manner described for Reach 1A, or that the weak Bay Mud be excavated to allow for fill construction directly on the stronger underlying clay layer. Each of these approaches is described below.

Floating Embankment - Based upon the estimated thickness and engineering properties of the Bay Mud, it is expected that the levee widening in this area could be accomplished using the system of grid reinforcement and lateral toe berm shown on Figure 9. For this approach, it is recommended that the levee side slope not exceed 2:1 (horizontal to vertical). Where the detonation site is to be developed by placing a wide pad of fill to about Elevation 7, it is expected that the toe berm could be eliminated providing that grid reinforcement is placed at the contact between the Bay Mud and fill to distribute the fill load and that the fill slope is flattened to 3:1 (horizontal to vertical).

Excavation of Bay Mud - As previously discussed in "Surface Features," a drainage ditch designed to have a 10 foot bottom width and 5 to 6 foot depth parallels the toe of the sludge lagoon levee. In lieu of floating the embankment fill as discussed above, the remaining Bay Mud could be removed by broadening and deepening the excavation along the levee toe as shown on Figure 10. All soft sediment and vegetation contained within the existing toe drain also should be removed. New levee fill placed directly on the stronger underlying clay layer should have a side slope inclination no steeper than 2:1 (horizontal to vertical).

At the time of the field reconnaissance for this study, water was observed to be standing in the toe drain to a depth which probably coincides with the existing groundwater level. Therefore, dewatering should be expected to permit fill placement in the dry. Since the excavation bottom will extend only a few feet below the groundwater level and the exposed soils probably will be clays and silts of relatively low permeability, it is anticipated that dewatering could be accomplished by pumping from sumps within the excavation area.

After the excavation dewatering has been completed, fill placement can begin. Although the soils exposed at the excavation bottom probably will consist of relatively stiff clays, it is anticipated that pumping subgrade conditions are likely to occur. In order to minimize the potential for pumping, it is recommended that the initial 1 foot of embankment fill be "track-walked" in place using lightweight equipment. The initial lift of fill should be dry of the laboratory optimum moisture content. Additional fill then could be placed and compacted to complete the levee section. If deep-seated pumping conditions develop, it could be necessary to "bridge" the subgrade using grid reinforcement and a layer of granular bank run material. These subgrade stabilization methods are discussed in further detail in the report section entitled "General Recommendations".

East of Station 80 - Along the remainder of the south side of the channel, it appears that the levee alignment will be underlain by alluvial deposits. It is expected that these soils will be capable of supporting new levee fills to the planned crest elevations using maximum side slopes of 2:1 (horizontal to vertical). This conclusion is based on the assumption that the areas to receive new fill first will be properly prepared.

Toe Drain Areas - At locations where the new levee fill will traverse existing drainage ditches or abut existing lagoon levees which have a toe drain along their toes, it is recommended that standing water, vegetation and all soft sediments be removed from the drain bottom areas before any new fill is placed. The extent of the excavation should encompass the minimum as-built dimensions of the drains discussed previously in this report. The recommended procedure is shown schematically on Figure 11.

Lagoon Interior - Where the new levee will pass through the interior of the existing sludge lagoon, it is expected that the native soils which underlie the sludge will appear soft and wet at the time the sludge is removed. In this area, it should be planned to aerate the areas to receive fill or bridge the subgrade by the methods described in "General Recommendations". An assessment of the lateral extent, depth and nature of soils contaminated by the sludge was beyond the scope of this investigation.

Open Area - As discussed in "Subsurface Conditions," uncompacted soil and trash fill was encountered in Exploratory Pit 9 in the open area between the west and east batteries of sludge lagoons. The trash fill was found to extend to a depth of about 4 feet; however its lateral extent was not determined. It is recommended that these materials and any other loose soil or trash fills which might be present along this portion of the alignment be excavated in their entirety to allow for construction of the new levee fill directly on the underlying native soils.

South Levee Settlement - Fill placed for the new south levee will cause compression of the underlying soils resulting in levee settlement. Estimated settlements for the portions of the south levee alignment discussed above are presented in the following table.

SOUTH LEVEE

Station Location(1)	70 to 80(2)	80 to 120(2)
Proposed New Fill Thickness (feet)	6 - 12	7 - 12
Compressible Layer Thickness (feet)	4 - 7 (Bay Mud/ Alluvium)	4 - 6 (Alluvium)
Estimated Ultimate Settlement (in.)	2 - 14	2 - 3
Estimated Time For 90% Settlement (yrs)	3 - 4	0.1 - 0.2

NOTES: (1) Refer to Figure 1b for alignment stations
(2) Centerline stationing of Bypass Channel

The estimated range of settlements presented above for levee construction from Station 70 to 80 are based on the assumption that the floating technique would be used. If the Bay Mud and/or compressible alluvial deposits were to be excavated to the stronger underlying materials, it is expected that post-construction settlements and the time for settlement probably would be comparable to the respective ranges listed for Stations 80 to 120.

North Levee Alignment

The subsurface conditions along the proposed levee alignment include areas of soil and trash fill, very soft tidal marsh and Bay Mud deposits, and young alluvial soils overlying Bay Mud or older clays. Specific recommendations pertaining to the sections of the levee alignment along which these conditions were encountered are discussed individually in the following paragraphs.

San Jose Scavenger Company Property - This short section of the proposed new levee alignment is underlain by soil and trash fill. Because of the considerable depth of this material (13 feet or more), excavation of it would not seem to be practical. Although the soil and trash fill is in a poorly compacted condition, it is expected that it would be capable of supporting a 4 to 5 foot high new levee with side slopes of 2:1 (horizontal to vertical). This recommended slope inclination is predicated on the assumption that the existing dredged levee fill would be removed and that key trenches would be provided below the new levee toes. Furthermore, it is recommended that the new levee be set back at least 10 feet from the top edge of the existing bank where it boarders the tidal marsh. The intent of this recommended set back is to help minimize the potential for undermining of the new levee toe should the relatively steep existing bank experience sloughing. These details are shown on Figure 12.

Tidal Marsh (Station 77 to 80) - Where the north levee is to traverse a portion of the existing tidal marsh area, it is expected that about 3 feet of tidal deposits and vegetation probably overlie 5 to 6 feet of Bay Mud. These estimated thicknesses are based on the comparative elevations of the tidal marsh (Elevation 4-1/2±) and adjoining former marsh area (Elevation 1) plus the approximate elevation at which very stiff native clay was encountered beneath the trash fill in Boring 9 (Elevation -5±). Thicker tidal deposits might be present in the borrow ditch from which material was dredged to form the narrow levee along the San Jose Scavenger Company property.

It is expected that the shallow tidal marsh deposits are in a near liquid state with little or no shear strength; whereas, the underlying Bay Mud probably has strengths comparable to those encountered in the salt pond area. New levee construction in this area could be accomplished by excavating or displacing the weak tidal deposits and Bay Mud using the methods described for Reach 1A; or, alternatively, by floating the embankment using grid reinforcement and a lateral toe berm. If the floating embankment alternative is selected, it is recommended that the general approach shown on Figure 5 ("open water") be used with the exception that the lateral berm be narrowed to 20 feet for a 3:1 (horizontal to vertical) levee side slope and 25 feet for a steeper 2:1 levee side slope.

Because of the extremely weak, almost liquid consistency of the tidal deposit, it is probable that it will be displaced by the new fill. For this reason, it is recommended that the working pad be advanced from the area nearest to Dixon Landing Road to minimize the potential for trapping of these deposits below the new levee section.

It also is recommended that the method of levee construction used in the tidal area be continued across the existing drainage ditch immediately to the east.

Tidal Marsh to Dixon Landing Road Bridge (Bypass Channel Station 80 to Coyote Creek Station 440) - Extending from the tidal marsh east, it is expected that the proposed levee alignment will be underlain by a variable thickness of young alluvial deposits over either Bay Mud or stronger alluvial clays. The performance of the sludge lagoon levees has demonstrated that 2:1 (horizontal to vertical) side slopes and fill heights equal to those presently proposed are feasible in the portions of the Reach 1B area which are underlain by alluvial deposits. However, where a relatively thin cover of alluvium overlies soft Bay Mud, somewhat flatter slopes are considered advisable to minimize the risk of shear failure in the Bay Mud. Since the sequence of alluvium over Bay Mud appears to predominate, it is recommended that the levee side slopes for this portion of the reach be designed for inclinations not to exceed 3:1 (horizontal to vertical).

Where the new levee will follow or traverse the existing interim flood control levee alignment, it is recommended that the existing loosely placed levee fill be removed prior to any new fill placement. For the section of the alignment which will parallel the existing creek, it is recommended that the levee toe be set back at least 10 feet from the top edge of the oversteepened creek bank. This recommended set back distance can be eliminated where the existing channel is to be backfilled. Additionally, it is recommended that existing surface fills present along this portion of the alignment be removed and key trenches be excavated beneath the levee toes as shown on Figure 12. It is estimated that the surface fills probably extend to about Elevation 5 in this area.

North Levee Settlement - Settlement estimates for the north levee alignment are presented on the following table. The estimates have been based on the thickness of compressible materials approximated by subsurface exploration, engineering judgement regarding their respective properties and the anticipated fill thickness along each portion of the alignment.

NORTH LEVEE

Station Location(1)	75 to 77(2)	77 to 80(2)	80 to 105(2)
Proposed New Fill Thickness (feet)	4 - 5	12 - 13	5 - 11
Compressible Layer Thickness (feet)	13 (Trash Fill)	3 (Tidal Deposits) over 6 (Bay Mud)	5-8 (Alluvium) over 0-11 (Bay Mud)
Estimated Ultimate Settlement (in.)	12 - 36(3)	18 - 24	2 - 7
Estimated Time For 90% Settlement (yrs)	Not Applicable	3 - 4	0.1 - 3

NOTES:
(1) Refer to Figure 1b for alignment stations
(2) Centerline stationing of Bypass Channel
(3) Settlement of levee fill bearing on trash fill could vary considerably depending on the nature and composition of the trash.

East Levee Alignment

The results of exploration in this portion of the Reach 1B area indicate that part of the east levee alignment will be underlain by several feet of young alluvial soils over soft Bay Mud, whereas Bay Mud is probably absent along the remaining part. It is expected that the soils in both areas will be capable of supporting new levees to the proposed design heights using maximum side slope inclinations of 2:1 (horizontal to vertical). Where existing fills are present along the south bank of lower Penetencia Creek and within the Milpitas Sanitation District facility area, it is recommended that these materials be removed to allow construction of the new levee on the underlying native soils.

After the sludge has been removed from the existing lagoon areas that are to be traversed by the new levee, it should be expected that drying or bridging of the subgrade soils will be needed before new levee fill is placed. An assessment of the lateral extent, depth, and nature of soils contaminated by the sludge was beyond the scope of this investigation.

East Levee Settlement - Estimated settlements due to new fill placed along the east levee alignment are summarized below.

	EAST LEVEE	
Station Location(1)	445 to 460(2)	460 to 480(2)
Proposed New Fill Thickness (feet)	5 - 9	10 - 12
Compressible Layer Thickness (feet)	5-8 (Alluvium) over 0-11 (Bay Mud)	5 - 7 (Alluvium)
Estimated Ultimate Settlement (in.)	2 - 6	2 - 3
Estimated Time For 90% Settlement (yrs)	0.1 - 3	0.1 - 0.2

NOTES: (1) Refer to Figure 1b for alignment stations
(2) Centerline stationing for Coyote Creek

Underseepage - As illustrated on the "Idealized Soil Profiles" presented as Figures 3b and 3c, most of the Reach 1B area is blanketed by a variable thickness of young alluvium which generally overlies either Bay Mud or older, stiff clay. The young alluvium consists primarily of low plasticity clays to silts and fine sands. Along the west end of the south levee alignment (from about Station 70 to 90) it appears that the fine sandy alluvium probably was excavated during preparation for the sludge lagoon construction such that the moderately to well-compacted levee fill bears on a relatively impervious clay foundation. It is expected that the potential for seepage either under or through these levees would be low.

Along the remainder of either side of Reach 1B, it is expected that portions of the new levee alignments will be underlain by sandy soils which either are exposed at the ground surface or have a thin top stratum of cohesive alluvium. Where present, these sandy deposits were found to extend to depths of about 5 to 12 feet below the existing ground surface or the base of the existing sludge lagoon levees. During flooding, the water level in the channel will create a hydraulic gradient under the levee causing seepage flow through the underlying pervious sandy stratum. Where the cohesive top stratum is either thin or absent, it is expected that the seepage will result in saturation of the ground surface or possibly some free standing water along the landward levee toe. Although the average hydraulic gradient under the levees would be relatively low, it is possible that some isolated areas of concentrated seepage could have exit gradients which are sufficiently strong to cause flotation or heaving at the toe and result in the development of sand boils. It is impossible to predict the locations where such concentrated seepage and sand boils might occur.

The need for seepage control measures could be evaluated after construction of the levees by observing the behavior of the landward toe areas during periods of moderate flow. If signs of seepage were observed, relief wells could be installed at the toe or a landside berm could be placed to lengthen the seepage path and reduce the exit gradient at those locations. This approach would result in a lower initial construction cost. However, there is an inherent risk that sand boils and piping could develop and damage the levee before remedial measures could be implemented.

Alternatively, the levee design could incorporate measures to help minimize the potential damaging effects of underseepage. In areas where a wet ground surface beyond the landward toe could be tolerated, the potential for sand boiling could be minimized by providing a keyway beneath the levee toe which would reduce the exit hydraulic gradient. The keyway should be equipment width (8 feet minimum), at least 3 feet deep, and should be backfilled with compacted, cohesive embankment fill.

The passage of seepage could be more completely blocked by overexcavating the sandy alluvium and providing a relatively impervious, compacted clay filled cutoff extending into the underlying relatively impervious older clays. This could be done using an equipment width (8 foot minimum) key trench positioned along the centerline of the levee alignment. Such a compacted clay fill cutoff probably would be feasible in areas where the sandy alluvium is less than about 5 feet thick, but would not be considered practical where it extends deeper and below the groundwater level. Other seepage control methods such as impervious channel-side blanket or landside berm also could be used to help increase the resistance to seepage entry and reduce the exit gradient. However, these methods would entail considerably more earthwork construction.

Considering the character of the foundation materials in the Reach 1B area, it is our opinion that a relatively impervious, earth-filled cutoff-wall constructed using slurry trench methods would provide a practical, relatively low cost means of blocking the passage of seepage beneath the levees. Recommendations regarding this method of seepage control are described below.

Slurry Trench Cutoff - Before the trench excavation is started, a working pad at least 25 feet wide should be constructed along the centerline of the proposed levee as shown on Figure 13. After stripping the area of vegetation, the surface soils should be scarified and recompacted to a minimum degree of compaction of 90 percent to a depth of 1 foot. Cohesive embankment fill then should be placed and compacted to provide a working pad surface at least 1 foot above the adjacent original ground surface. The purpose of the working pad is to help minimize the potential for a seepage path developing at the contact between the base of the new levee fill and the top of the underlying slurry trench cutoff.

After the working pad has been constructed, it is recommended that a trench at least 2 feet wide be excavated through the pervious sand stratum into the underlying relatively impervious clay layer (alluvial clay or Bay Mud) as a bentonite and water slurry is pumped back into the trench. The purpose of the bentonite slurry is to enable the excavation of a vertical sided trench below the groundwater level and through the pervious stratum without the need for shoring and dewatering. The trench excavation should extend at least 2 feet into the underlying clay layer.

Once the recommended penetration has been achieved, a homogeneous mixture of soil and bentonite slurry should be placed to backfill the trench and form the cutoff wall. Material used for backfilling the slurry trench can consist of the excavated trench materials provided that a minimum of 30 percent by dry weight passes the U.S. Standard No. 200 sieve. At least 1 percent bentonite slurry should be mixed with the excavated materials to achieve a consistency similar to wet concrete with a 4 to 6 inch slump. Details regarding the construction of the slurry trench cutoff are shown on Figure 13.

Along most of the Reach 1B area, it is expected that the recommended penetration through the sandy alluvium and into the underlying alluvial clay or Bay Mud would require an excavation depth of about 7 to 10 feet below the natural ground surface. Complete penetration of the sand stratum near the east end of the south levee alignment probably would require an excavation depth of 14 feet or more. A better estimate of the depth and variability of the sand layer along each of the Reach 1B levee alignments could be made by conducting a series of cone penetrometer soundings prior to slurry trench construction. Additional subsurface exploration would also help delineate areas in which a slurry trench might not be needed, thereby possibly decreasing the cost of seepage control measures.

A slurry trench cutoff also could be used to minimize underseepage through the soil and trash fill found in the San Jose Scavenger Company property. Based on the conditions encountered in Boring 9, it is estimated that a cutoff depth of about 15 feet might be needed. However, it is recommended that more detailed study of the extent and composition of the trash fill be considered prior to designing a slurry trench cutoff for this portion of the Reach 1B levee alignment. The final cutoff depths should be determined by the Geotechnical Engineer during construction.

The procedures recommended above assume that the slurry trench cutoff would be incorporated as part of the levee design. However, it is conceivable that slurry trench cutoff also could be installed where underseepage might develop at isolated locations beneath the completed levee. For this application, it is recommended that the cutoff depth, lateral extent and location (either channel side or landward side of the levee) be determined on an individual basis for the specific subsurface soil and seepage conditions.

Dixon Landing Road Floodwall

Design Lateral Pressures - A cantilever wall serving as a floodwall should be designed to resist the water pressure exerted on the front of the wall during periods of high flow, as well as the long-term lateral earth pressures imposed on the back of the wall by the backfill and any surcharge or traffic loads. It is recommended that the lateral earth pressure acting on the back of the wall be idealized by an equivalent fluid pressure weighing 40 pcf. Auto and light truck traffic loads can be simulated by a surcharge pressure equivalent to an additional height of 2 feet of soil.

Foundation Support - It is expected that the subsurface conditions along the proposed floodwall alignment consist of a heterogeneous mixture of soil and rubble fill materials. It is considered inadvisable that shallow foundations be supported directly on these materials. To use a spread footing foundation to support the wall, it is recommended that the existing fill be excavated and replaced with engineered fill to a minimum depth equal to 4 feet or one-half the footing width, whichever is deeper. The base width of the excavation and engineered fill construction should extend to a plane projected downward from the bottom edges of the footing at an inclination of 1:1(horizontal to vertical); but in no case should the width be less than that which will accommodate large segmented wheel compaction equipment.

Footings bearing on engineered fill should be designed so that maximum bearing pressures do not exceed 2,000 psf due to dead loads, 3,000 psf due to dead plus live loads and 4,000 psf due to all loads including wind or seismic. Resistance to sliding can be assumed to be developed by friction along the base of the wall footing. For design purposes, a coefficient of friction of 0.40 can be assumed.

Roadway Fill Alternative - As previously discussed in "Surface Features" the existing road surface along the proposed floodwall alignment is positioned at about Elevation 10-1/2 to 11. In lieu of constructing a floodwall along this area, it is suggested that the placement of fill to raise the road surface to Elevation 13 be considered.

PROPOSED CHANNEL IMPROVEMENTS

Along the Reach 2 portion of the alignment, it is planned to retain the existing creek channel and construct new levees at varying distances from the creek banks to provide a constrained flood plain as shown in plan view on Figure 1c. In order that the trees and natural vegetation which line the creek can be preserved, it is intended to provide a minimum levee set back from the creek which would approximately coincide with the dripline of the trees.

In general, the flood plain width along Reach 2 will vary from about 500 to 900 feet. The exception will be at the downstream end of the reach where the channel will narrow to about 300 feet as it passes between the northeast corner of the sludge lagoons of the San Jose-Santa Clara Pollution Control Plant and the Milpitas Sanitation District facility on the opposite creek bank. At this location, it is planned to deepen the channel and construct a rock lined drop structure to provide a hydraulic control feature to raise the water surface. Preliminary drawings prepared by the District indicate that the low flow portion of the channel would pass near the toe of the northeast corner of the sludge lagoon levee. The existing channel would be both widened and deepened with a new bottom depth at about Elevation 2 and proposed cut slope inclinations of 2:1 (horizontal to vertical). It is understood that revisions to that preliminary design probably will be made. However, specific details regarding a revised drop structure design were not available at the time of this writing.

Extending upstream for a distance of about 2,500 feet along the west side of the channel, the new levee will adjoin the existing sludge lagoon levee. Where the creek passes beneath the existing bridges at Alviso-Milpitas Road and Highway 237, it is planned to improve the flow capacity by providing a concrete lined channel with a stepped trapezoidal shaped configuration. Concrete lined channel slopes with 1-1/2:1 (horizontal to vertical) inclinations have been proposed. It is understood that these design details also are preliminary and subject to revision.

In order to provide at least 3 feet of freeboard above the 1 percent design flood level, the new Reach 2 levees will have a profile which will gradually rise from about Elevation 19-1/2 at the downstream end to about Elevation 27-1/2 upstream of Highway 237. This design profile will require levee fills on the order of 10 to 14 feet above the existing grade. It is planned that much of the levee fill will be derived from channel benches excavated within the flood plain limits. These excavated channel benches will be graded to slope toward the existing creek. The crest width of the proposed new levees will be 18 feet.

SITE AND GEOTECHNICAL FEATURES

Surface Features

Topographic information provided by the District (June, 1983) indicates that the natural ground surface along the Reach 2 portion of the project alignment slopes upward to the southeast from about Elevation 10 near the Milpitas Sanitation District facility to about Elevation 18 near Highway 237. Adjacent to the creek banks, the ground surface is typically somewhat higher. This is particularly evident upstream of the McCarthy packing plant where the creek is flanked by existing flood control levees which gradually increase to heights of more than 8 feet above the natural grade near the Highway 237 bridge. Extending from the top of the banks to the water surface, the creek side slopes are nearly vertical in areas where recent sloughing and undermining have occurred and are relatively shallow at other locations.

Review of the topographic information also indicates that the crest of the existing sludge lagoon is positioned slightly below Elevation 16 along the portion where it will be adjoined by the new west levee. As previously discussed in the Reach 1B section of this report, the existing sludge lagoon levees were designed with 2:1 (horizontal to vertical) slopes and a 5 to 6 foot deep drainage ditch along the toe. The base of the levee now appears much flatter and the toe drain is a mere swale overgrown with weeds. It appears that these changes from the as-built conditions are the result of sediment deposition during flooding in this area.

Subsurface Soil Conditions

Borings 16 through 32, P-3 through P-6 and 6 exploratory pits were made along Reach 2 to supplement existing subsurface data from previous studies. The locations of the explorations are shown on Figure 1c and the exploratory procedures used during this study are described in Appendix A. An "Idealized Soil Profile" (Section D-D') has been prepared by direct interpolations between these explorations along the east side of the alignment to illustrate the conditions encountered along Reach 2. This soil profile is presented on Figure 3d.

The subsurface conditions encountered along Reach 2 were found to consist primarily of clayey and sandy alluvium with some gravelly zones. At some locations where the creek banks have been over-topped during recent flooding, a thin layer of fine sandy silt to silty sand covers the ground surface. This condition is most evident along the downstream portion between the west creek bank and the existing sludge lagoon levee. At locations further upstream, poorly to moderately compacted levee fills consisting of a variety of soils from clays to gravel flank the creek banks. Excepting these deposits, the native surficial soils consist of low plasticity clayey silts to silty clays which exhibit plasticity indices in the range of 6 to 15. These deposits are medium to stiff in consistency near the ground surface and decrease in strength with depth and increasing moisture content. The thickness of the surficial layer is about 3 to 5 feet in the downstream part of the reach and gradually increases to more than 10 feet upstream of Highway 237.

In most of the explorations, the predominantly cohesive surface soils were found to be underlain by a layer of loose to medium dense sandy silt to silty sand. These cohesionless deposits generally were found to be in a saturated condition with the top of the layer approximately coinciding with the groundwater level in the explorations. The layer is variable in thickness along the reach but appears to be somewhat more prevalent on the east side of the creek than on the west. In the downstream part of the reach where the new west levee will abut the existing sludge lagoon levee, similar loose to medium dense cohesionless deposits were logged by Dames & Moore (1971) within about the same depth intervals during their exploration of the sludge lagoon area. A deeper silty sand layer about 11 feet thick was encountered beneath well compacted levee fill, in Boring 17 of this study, where the sludge lagoons are in close proximity to the creek; however, cohesionless deposits were not detected in the other borings of this study which were drilled further south either through or adjacent to the existing levee. It is speculated that the shallower sandy materials probably were removed during the grading operations. Recent explorations by Peter Kaldveer and Associates for additions to the sludge lagoons of the San Jose-Santa Clara treatment plant reportedly did not detect the presence of cohesionless materials. This further demonstrates the variable nature of the shallow alluvial soils in this area.

In approximately the downstream one-half of the reach, the layer of cohesionless sands and silts is underlain by stronger silty clays which grade from high to low plasticity with increasing depth. Further upstream, a sequence of clay overlying deeper silty sand to sandy gravel generally was encountered; although in Boring 25, the clay layer was absent and the cohesionless sands and silts were underlain by sandy gravel.

Conditions differing from those described above were encountered in some of the other explorations made along Reach 2. In Boring 23, which was drilled behind the McCarthy packing plant facilities, an 8 foot thick layer of soft Bay Mud was encountered about 20 feet below existing grade. Topographic features which might be suggestive of a slough are not evident from the available elevation contours; nor is a slough shown on the historical maps of the mid-1800's. However, tonal differences are evident on the previously referenced 1928

Leslie Salt Company aerial photo of this area which suggest that a relic slough or buried channel probably extended in a northwesterly direction through the McCarthy property toward the bay. Dames & Moore (1971) also encountered a 17 foot thick layer of soft to medium stiff Bay Mud in their boring DM-52 which was drilled within the slough alignment suggested by the old aerial photo. That alignment is approximated on Figure 1c.

Further upstream, the general alluvial sequence found in most of the explorations was interrupted in Boring 28 which was drilled just downstream of the east abutment of the old Alviso-Milpitas Road bridge. Based on a review of "Farm Map Number Two" of Santa Clara County surveyed in the mid-1800's (Historical Atlas, 1973), it appears that a channel tributary to Coyote Creek previously extended to the east along the old road alignment. It is likely that the conditions encountered in Boring 28 represent old creek bed deposits or possibly old fill. In any event, these materials were found to be somewhat more sandy than the natural conditions logged at other locations and exhibit moderate strengths and densities. A rough approximation of that old channel alignment also is shown on Figure 1c.

Groundwater Conditions

Free groundwater was encountered in most of the explorations made in the Reach 2 area. At the time of the field investigation, groundwater was encountered as shallow as 3 1/2 feet below the site surface (approximate Elevation 7) in the downstream area and appeared to approximately coincide with the water level in the creek. Further upstream, the groundwater was recorded at increasingly deeper depths but similar elevations (approximate Elevation 4 to 5). The groundwater levels recorded during the field exploration program are indicated on the Exploratory Logs.

It is understood from Mr. Kenneth Reiller of the District, that groundwater measurements at piezometer Borings P-1 through P-6, which were begun in July, 1983, have shown seasonal groundwater level fluctuations with a rise of as much as 4 feet from the levels measured at the time of drilling.

DISCUSSION AND RECOMMENDATIONS

General

The alluvial deposits encountered along the Reach 2 portion of the project alignment are generally of moderate strength near the surface and become stronger with increasing depth. Where the west levee will adjoin the existing sludge lagoon levee, the existing levee fill was found to be in a moderately to well compacted condition. It is expected that these foundation materials will be capable of supporting the loads that will be imposed by the new levee fill.

Levee Slope Inclinations

Based upon the subsurface conditions encountered along the reach and the anticipated new fill thicknesses, it is recommended that the maximum levee side slopes be designed for inclinations not to exceed 2:1 (horizontal to vertical). This recommended levee slope inclination is, in part, based on the assumption that the areas to receive new fill will be properly prepared and that the fill will be constructed to the requirements of engineered fill. Recommendations regarding general site preparation and earthwork construction are presented in the report section entitled "General Recommendations".

Along the existing sludge lagoon levee, it is recommended that all loose sediment and vegetation be removed from the base of the levee and from within the toe drain area before any new fill is placed. The extent of the removal should encompass the minimum as-bult dimensions of the drains. The recommended procedure is shown schematically on Figure 11.

Set Back

As presently planned, the new flood plain levees will be positioned no closer than the dripline of the existing trees which line the creek banks. Although the intent of this design is to preserve the natural vegetation, this set back also will help minimize the potential for undermining of the new levee toe in areas where the existing creek banks have become oversteepened due to sloughing. However, if it is planned to position the toe of the new levee inside of the dripline in some areas, it is recommended that it extend no closer than a 2:1 (horizontal to vertical) plane projected up from the edge of the existing channel bottom or 10 feet from the top edge of the bank, whichever is greater.

Settlements

It is estimated that the post construction levee settlement along most of the Reach 2 alignment will be less than 3 inches. Where the new levees will cross the old slough, it is expected that somewhat larger settlements could occur due to the more compressible nature of the Bay Mud contained in the old slough. It is estimated that the ultimate settlement at these isolated locations might range from about 6 to 9 inches. These estimates are based partly on the Bay Mud thicknesses which were logged in Borings 23 and DM-52, respectively, as well as existing information regarding the general compressibility characteristics of those deposits.

Reusability of On-site Soils

The near surface soils encountered along Reach 2 consist primarily of low plasticity clayey silts to silty clays. These materials are considered to be suitable for reuse as new levee fill because of their high strength and relatively impervious nature when placed in a well compacted condition. The coefficient of permeability (k) estimated by variable head permeability tests performed using samples of these materials was found to range from about $1x10^{-6}$ to $9x10^{-8}$ cm/sec. These coefficients of permeability were estimated for samples compacted in the laboratory and a drive sample of the existing sludge lagoon levee fill, respectively. The results of the tests are summarized on Table A-2.

It is expected that more pervious fine sandy silts and silty sands, such as those deposited in the downstream area during recent flooding, will be encountered at many locations along the alignment. It is recommended that these materials either be thoroughly blended with an equal portion of clay soils or be placed on the landward one-third of the levee. It should be anticipated that some materials may be in a near saturated condition and will require spreading, drying, and discing prior to their placement as levee fill.

Shrinkage

In order to help estimate the volumetric change which would be expected to occur between excavation and emplacement of the on-site soils as levee fill, samples were compacted in the laboratory to establish their respective maximum dry unit weights. The results of these tests are presented on Table A-1. Comparisons of the densities needed to achieve an average degree of compaction of 92 percent then were made with the insitu densities estimated during the laboratory testing program. Based upon these comparisons, it is estimated that an average shrinkage of about 16 percent should be expected. This percent shrinkage means that one cubic yard of excavated material would occupy 0.84 cubic yards as compacted levee fill. It is recommended that 16 percent shrinkage be assumed for earthwork estimating purposes.

Channel Bench Excavations

As discussed under Project Description, it is planned to excavate soil from areas adjacent to the existing channel in order to provide greater channel capacity. Also, the material will provide a source of fill material for new levees. At the time of the field exploration program, the free groundwater level was encountered as shallow as Elevation 7, or about 3-1/2 to 4 feet below the existing ground surface at some locations in the downstream part of the reach. Moving upstream the groundwater table appears to remain at a relatively constant elevation but the depth below existing grade increases. Soils in a near saturated condition also were encountered in a zone of capillary rise generally extending about 1-1/2 to 2 feet above the free groundwater level. It is expected that excavation of the soils above the zone of capillary rise could probably be accomplished using scrapers. However, as the excavation deepens, pumping and rutting of the wet, very silty materials are likely to limit or prohibit the use of rubber tire equipment. Excavation below this level will likely require a period of aeration to develop a crust which could again support scraper loads or, alternatively, the use of equipment with flotation tracks. Although the selection of the bench elevation is primarily a hydraulic consideration, the depth of practical excavation should be weighed.

Underseepage

Explorations made along Reach 2 indicate that new levees constructed directly on the existing grade probably will rest on a thin surficial layer of cohesive alluvium of relatively low permeability which grades to a more pervious substratum of sands and silts with some gravelly zones. In some areas the cohesive top layer might also be absent. Where the pervious sandy stratum is present, it is expected that underseepage probably will occur during periods of flooding.

Following the earthquake of 1906, sand boils were reported in the fields along Reach 2 (Youd and Hoose, 1978). Although these sand boils were the result of excess pore pressures developed during earthquake ground motions, they do give an indication of the potential for this phenomenon. Sand boils also were observed by the District on March 3, 1983, along the east bank of the creek upstream of Highway 237 near Agnews during a period of high flow (District Memorandum, 1983).

Based upon the subsurface conditions and reported occurrences of sand boiling described above, it appears likely that sand boils also could develop along Reach 2 as a result of levee underseepage. As previously discussed for Reach 1B, the areas requiring seepage control measures might be identified after construction by observing the behavior of the landward toe of the levees during moderate flow periods; or, alternatively, the levees could be designed with measures to minimize the potential for seepage through the foundation materials.

It is our opinion that the construction of an impervious, earth-filled cutoff wall by the slurry trench method, previously recommended in this report for Reach 1B, also would provide a practical means of blocking underseepage through the pervious substratum in Reach 2. The trench excavation also should have a minimum width of 2 feet and should extend at least 2 feet into the underlying, stiff alluvial clays (where present) as shown on Figure 11. In the downstream portion of the reach, north of Boring 20, it is expected that this would require an excavation depth of about 7 feet. Further upstream, the sandy stratum becomes thicker in some areas but also appears to be discontinuous in others. It is estimated that a trench depth of about 10 to 12 feet probably would accomplish the recommended penetration into the underlying clay layer except in areas which are underlain by deeper sand or gravel layers. Such conditions were encountered in Boring 25, where the underlying clay layer was absent; and in Boring 28, where it appears that an old creek channel probably underlies the east levee alignment near Alviso-Milpitas Road. The limits of these apparently anomalous conditions, as well as the depth of the cutoffs, might be better estimated by a series of cone penetrometer soundings made at more closely spaced intervals along the levee alignment prior to construction. The final cutoff depths should be determined by the Geotechnical Engineer during construction.

In areas where the pervious materials are not completely penetrated by the cutoff, relief wells could be installed to relieve excess pressure. The need for relief wells should be assessed following installation of the cutoff trench and/or observation of the performance of the levee and cutoff trench.

Drop Structure

The explorations made in the vicinity of the proposed drop structure also disclosed a sequence of recent, low to moderate strength clayey and sandy alluvium overlying stronger, older clays. The sandy substratum was found to increase in thickness from about 1 foot at the east side of the creek (Boring 16) to about 11 feet beneath the existing sludge lagoon levee fill on the west side (Boring 17). Excavation to broaden and deepen the channel to the preliminary design depth of Elevation 2 would be expected to position much of the channel bottom in the sandy substratum and as much as 4 feet below the free groundwater level encountered at the time of drilling. If similar groundwater conditions prevail at the time of construction, dewatering would be needed to minimize the potential for undermining of the existing sludge lagoon levee toe or the development of a quick condition in the excavation bottom due to the force of groundwater seepage. With proper groundwater control and the placement of rock lining designed with a filter layer to prevent subsurface erosion due to piping, it is expected that a proposed 2:1 (horizontal to vertical) channel cut slope inclination would be feasible. Specific geotechnical recommendations should be developed when the drop structure design has been finalized.

Concrete Lining Under Highway 237 Bridge

The subsurface conditions encountered along the creek banks in the vicinity of the Highway 237 bridge and old Alviso-Milpitas Road bridge were found to range from predominantly sandy and gravelly soils with thin clay layers (Boring 29) to almost entirely clayey soils (Borings 30 and 31) within the full depth explored. These deposits generally exhibit moderate strengths and would be expected to stand at 1-1/2:1 (horizontal to vertical) inclinations would be feasible providing that they are concrete lined. Drainage through the lining should be provided to prevent the development of hydrostatic uplift pressure. Based on discussions with Mr. Kenneth Reiller, it is understood that a standard District weep hole design probably would be used for the channel lining in this area.

GRADING

Site Preparation

Before any fill is placed to construct the new levees, the existing site surface should be properly prepared. With the exception of the below water portion of the salt pond, it is recommended that the surface of areas to receive fill should be cleared and grubbed in accordance with the provisions of Section 16 of the Caltrans Standard Specifications.

Overexcavation

After the areas to receive fill have been properly cleared and grubbed, it is recommended that existing loose fills or soft deposits present in areas such as the existing drainage ditches along the toes of the sludge lagoon levees be removed to the underlying native soils. The exception would be at locations where the embankment fill is to be floated over existing soft deposits. Where the new levee alignments will pass through existing sludge lagoons, the sludge should be removed to the underlying soils. An assessment of the sludge depth and the extent of contamination of the underlying soils was beyond the scope of this investigation. Areas expected to contain loosely placed soil fills or buried trash fill should be excavated as discussed previously in the report for each of the alignment reaches. Key trenches also should be excavated at the recommended locations below the new levee toes.

Where demolition is anticipated to remove existing structures within the project limits, it would be desirable, from a geotechnical standpoint, that the resulting excavations extend to undisturbed native soils and be left in an unbackfilled condition until mass grading is begun.

Subgrade Preparation

Following the completion of the stripping and overexcavation of loose fills, the subgrade should be scarified, moisture conditioned and compacted to a depth of at least 6 inches prior to the placement of new fill. Where fill will be placed along the existing salt pond levee, it is recommended that the irregular mounds and blocks of dredged Bay Mud be broken down with a lightweight bulldozer or tractor. The exposed soils then should be disced to promote drying. After these exposed materials have been moisture conditioned to near the optimum moisture content, they should be compacted to a minimum degree of compaction of 85 percent by track-walking. The base of key trench excavations also should be scarified, moisture conditioned and track-walked prior to the placement of new fill materials.

At many locations, the subgrade surfaces will be positioned below the free groundwater level or in areas which have been subjected to standing water (or sludge). The exposed materials are expected to consist of low plasticity clays, silts and fine sands.

Control of the water in most shallow excavations in these materials probably could be accomplished by sloping them to drain and pumping from sumps within the excavated areas. After dewatering, some of the subgrade surfaces might be stable enough to support construction equipment; whereas, other areas might develop pumping or rutting.

In order to minimize the potential for pumping subgrade conditions, it is recommended that the first lift of fill be "track-walked" in place to a thickness of about 1 foot using a bulldozer or other track-mounted equipment. The initial lift should be dry of the laboratory optimum and compacted to a minimum degree of compaction of 85 percent. Degree of compaction is the ratio of the in place density of the constructed fill to the maximum dry density determined by Test Method 216-F of the Caltrans Materials Manual, 1973 edition. Rubber tired equipment, such as scrapers or bottom dump trucks should not be permitted to traverse the area.

In the event that deep seated pumping persists, it is recommended that the unstable subgrade areas be "bridged" using a combination of grid reinforcement (Tensar Geogrid SS-2 or equivalent) or stabilization fabric (Mirafi 500X or equivalent) covered by a layer of granular, bridging material. The bridging material should consist of "bank run" or "crusher run" materials conforming to the general gradation limits, recommended previously in the Reach 1A section of this report, for the construction of a working pad in the salt pond area.

After the grid reinforcement or reinforcing fabric has been placed on the exposed subgrade surface, the bridging material should be track-walked into place over the fabric. It is estimated that an 18 inch thick layer of bridging material probably will be needed. Rubber tired equipment should not be permitted to traverse pumping areas until placement of the bridging material and grid reinforcement have been completed. Then additional fill can be placed and compacted to provide to desired finished grades.

Placing and Compacting Fills

All levee fills should be placed in thin lifts and compacted to a minimum degree of compaction of 90 percent as set forth in Sections 19-5 and 19-6 of the Caltrans Standard Specifications. Fills placed on slopes steeper than 6:1 (horizontal to vertical) should be benched into the adjacent soils to provide a bond at the interface. Loose soils on the surface of existing slopes should be excavated as new fill is placed. Benching is expected to be necessary where new levees will adjoin the existing salt pond and sludge lagoon levees. It is recommended that the minimum extent of benching (measured perpendicular to the slope face) be 1 foot. The adequacy of this should be confirmed during construction.

Materials

On-Site - The existing salt pond levee is comprised primarily of dredged Bay Mud with traces of sand and shells. Where it is planned to remove portions of this existing levee, the materials could be reused as new levee fill providing they are properly processed to achieve a moisture content near the optimum prior to placement. The drying of materials with excessive moisture content should be in accordance with the requirements of Section 19-6 of the Caltrans Standard Specifications. The existing sludge lagoon levees and interim levees in the Reach 1B area also consist primarily of cohesive soils which appear to be reusable as new levee fill. Along Reach 2, some of the existing levees which parallel the upstream banks appear to consist of cohesionless soils. It is expected that some selectivity will have to be exercised if these materials are to be considered for reuse as possible new levee fill. As discussed in the previous section of this report, it appears that the majority of the native soils within the proposed channel bench areas can be reused as levee fill providing that the more sandy deposits are blended with clay soils or utilized only along the landside of the levee.

Import -

General Levee Fill - If import materials are needed for levee construction, it is recommended that they consist of a soil or soil-rock mixture free from organic matter or other deleterious substances. The material should contain at least 30 percent by weight of clayey soils (passing the U.S. Standard No. 200 sieve). The clayey soils should be of low to medium plasticity as classified by the Unified Soil Classification System having a plasticity index within the range of 10 to 30. The material should not contain rocks or lumps over 6 inches in greatest dimension and should not contain more than 15 percent greater than 2 1/2 inches. Sand and gravel size particles should comprise no more than 70 percent of the soil-rock mixture. It is anticipated that materials meeting these general requirements are available from local quarries or borrow pits.

Bank Run - Bank run material for use with the displacement and floating embankment methods, or for bridging soft subgrades, should consist of a reasonably well graded mixture of sand, gravel and cobbles. The following gradation limits are recommended as a general design guideline.

Sieve Size	Percent Passing
12 inch	100
6 inch	70-100
2 inch	40-100
#4	20-50
#200	0-20

It is anticipated that materials meeting these requirements can be obtained from local quarries. The Geotechnical Engineer should review potential sources of bank run material prior to construction.

SCOUR AND EROSION PROTECTION

Scour and erosion protection should be provided with high flow velocities or wave action could cause erosion of the slope surface. The slope surface should be examined periodically and repaired as necessary to maintain the design embankment section. Analysis of erosion protection is beyond the scope of the study.

ASSESSMENT OF LEVEE BEHAVIOR DURING EARTHQUAKES

INTRODUCTION

Performance of the proposed Coyote Creek levees during earthquakes is dependent upon the earthquake ground motions that may occur in the area, as well as the physical properties of the levees and soils beneath the levees. An assessment of earthquake ground motions and of the subsurface conditions has been made to assist in evaluating the potential for deformation of the levees during earthquakes. This report section presents the results of the levee stability evaluation and includes specifically the following information:

- Identification of seismic sources in the area and estimation of site ground motions;
- Evaluation of site liquefaction potential based on field data, analyses, and historic records;
- Summary of methodology of levee deformation analyses;
- Summary of subsurface soil conditions pertinent to seismic stability of various portions of the levee alignment, followed by an assessment of levee behavior, and a discussion of possible mitigation measures.

The conclusions of this section have been considered in the development of the recommendations and opinions contained in the preceeding report sections.

SEISMIC SOURCES AND ESTIMATED GROUND MOTIONS

The proposed Coyote Creek improvements are located approximately 13 miles east of the San Andreas fault, one mile west of the Hayward fault, and 5 miles west of the Calaveras fault. The behavior of proposed levees has been evaluated for estimated maximum earthquakes and 100-year earthquakes on the San Andreas fault and the Hayward fault. Maximum and 100-year earthquakes on these faults are considered to be more critical to levee stability than maximum and 100-year earthquakes on the Calaveras fault. The estimated magnitudes of maximum and 100-year earthquakes and resulting site ground motions for which levee behavior has been assessed are summarized below.

	Earthquake Magnitude (Ms)		Peak Ground Acceleration (g's)	
	Maximum Credible Earthquake	100-year Earthquake	Maximum Credible Earthquake	100-year Earthquake
Hayward	7	6 1/4	0.50	0.44
San Andreas	8	6 1/2	0.30	0.15

For purposes of estimating site peak ground accelerations, it has been assumed that earthquake fault rupture would occur on portions of the faults closest to the site for each earthquake considered. The peak accelerations represent median estimates of peak ground acceleration on level ground (free field) for areas underlain by alluvial soils consisting of clays and sands. These estimates are probably somewhat conservative for parts of the levee alignment underlain by Bay Mud. It is expected that the accelerations in the levees would not differ greatly from the level ground (free field) values, and, therefore, the peak accelerations summarized above have been assumed to occur in the levees as well as at the level ground surface.

ASSESSMENT OF LIQUEFACTION POTENTIAL

During an earthquake, shallow, saturated, and loose granular soil deposits (sands, silts, and gravels) may experience an increase in pore water pressure and partial or complete loss of strength, depending on the intensity of ground shaking. The phenomenon of pore pressure buildup and loss of strength is known as liquefaction.

Sandy and silty strata were encountered at depths of a few to several feet over an extended area in various reaches of the proposed levee alignment. Behavior of these shallow granular soils during earthquakes has an important influence on stability and deformation of the overlying levees. Greater damage due to the displacement of the levees are expected if the underlying soils build up pore water pressure and liquefy. Therefore, an evaluation of the potential for liquefaction of these shallow sands and silts was made.

A measure of the liquefaction potential of sandy and silty soils is the standard penetration test (SPT) blow count measured in field borings. The SPT blow counts measured in the soil borings along the levee alignment were analyzed using the current, empirically-based liquefaction evaluation procedure of Seed and Idriss (1982) for the site ground motions estimated during maximum and 100-year earthquakes. This analysis indicates that during either maximum or 100-year earthquakes, there is, in general, a high potential for liquefaction of the shallow sands and silts.

Liquefaction has been documented to have occurred along this portion of the Coyote Creek during the 1906 San Francisco earthquake. Evidence of liquefaction included sand boils and lurching of the ground toward the creek (Youd & Hoose, 1978).

METHODS OF LEVEE DEFORMATION ANALYSES

Analyses were made to obtain an indication of the approximate amount of deformation that the proposed levee might be expected to undergo during earthquakes. The analyses were made using the procedure of Makdisi and Seed (1978) in which the amount of displacement is related to the earthquake-induced accelerations and the accelerations required to cause yielding of potential sliding masses. The earthquake-induced accelerations were taken equal to the values summarized previously for maximum and 100-year earthquakes. The yield accelerations were estimated by performing pseudo-static analyses of potential sliding masses using both circular arc and wedge stability analysis methods.

A general finding of the deformation analyses is that maximum earthquakes on either the San Andreas or the Hayward fault are expected to induce the same order of magnitude of deformations. A similar result was found for 100-year earthquakes. Therefore, in the following discussions, levee behavior is assessed for maximum earthquakes and 100-year earthquakes without reference to the causative fault.

ASSESSMENT OF LEVEE DEFORMATIONS

In the following paragraphs, the potential for levee deformations is assessed for the following portions of the levee alignment that are characterized by somewhat different subsurface conditions: Reach 1A - salt pond area; Reach 1B - south levee alignment along the sludge lagoons; Reach 1B -

north and east levee alignment along the extension of Dixon Landing Road and extending to the Milpitas Sanitation District facility; and Reach 2 - Milpitas Sanitation District facility to Highway 237. Idealized soil profiles for these reaches are shown in Figures 3a through 3d. For convenience, a summary of the subsurface soil conditions is reiterated below for each of these reaches of the levee alignment, followed by an assessment of levee behavior during earthquakes, and a discussion of possible mitigating measures which might improve levee behavior.

Reach 1A - Salt Pond Area

This reach of the levee alignment, between Borings 1 and 7, will be constructed through the salt pond area. Sections of the levee will be in open water as well as adjoining the existing salt pond levee in other locations.

Subsurface Soil Conditions - The idealized subsurface conditions along this reach are illustrated on Figure 3a. The soils are typically very soft Bay Mud underlain by predominantly stiff silty clay. The thickness of Bay Mud varies between 5 and 17 feet with a typical value of about 10 feet. A thick layer of silty sand was encountered beneath the Bay Mud in Boring 1; however, evidence of this sand layer was not detected in Borings 2 to 7.

Estimated Levee Behavior - For a floating levee constructed in open water, the portion of the levee fill placed below the water level could possibly build up pore water pressure and liquefy during earthquakes, which would govern levee behavior in this case. However, because the thickness of levee fill below water will be relatively thin (only about 2 to 5 feet initially) and is anticipated to consist of a fairly permeable imported bank run fill meeting the gradation requirements described previously in this report, it is likely that significant pore pressures would not develop in the fill. Thus, behavior is probably goverened by the shear strength of the Bay Mud beneath the fill. Since the Bay Mud is expected to gain in strength as it consolidates under the load of the levee fill, the behavior will be a function of the time after levee construction. In the unlikely event that an earthquake were to occur shortly after construction (with little Bay Mud consolidation and associated strength gain), levee displacement would be unlikely to exceed about 10 feet during a maximum earthquake; during a 100-year earthquake, the displacement would be unlikely to exceed about 3 feet. On the other hand, if the Bay Mud were fully consolidated (expected in about 10 years for 90% of the area under the levee) a displacement of about 3 feet is estimated for a maximum earthquake and less for a 100-year earthquake.

The behavior of levees constructed either by predredging or displacing the Bay Mud depend greatly on the permeability of the fill material placed below water. If the fill is pervious enough not to build up significant pore water pressure during shaking, such as the granular bank run material described previously in this report, then displacements of about 1 to 2 feet during maximum earthquakes and less for 100-year earthquakes are estimated. However, displacement in excess of 10 feet could occur if the underwater fill were not sufficiently pervious such that significant pore water pressures developed during earthquakes.

For levees constructed adjacent to and abutting the existing salt pond levee it is anticipated that a substantial amount of the levee fill will be placed below the water level in the existing borrow ditch. Therefore, the behavior of these levees also depends mainly on the potential for pore pressure build up in the submerged portion of the fill. Thus, behavior is estimated to be similar to that for displacement-type levees as discussed above.

In the vicinity of Boring 1, levee behavior may be governed by the potential for liquefaction of the sand beneath the Bay Mud. Liquefaction could result in several feet of levee displacement during either maximum or 100-year earthquakes.

Mitigating Measures - As discussed above, deformation of the levees during an earthquake may occur due to shearing through the Bay Mud or liquefaction of the granular fill placed underwater, or a combination of both. The potential for deformations due to shearing through Bay Mud decreases with time after construction, such that in about 10 years, earthquake-induced displacements due to a maximum earthquake would be unlikely to exceed about 3 feet. Reduction of deformation due to shearing through Bay Mud could be achieved by using flatter levee side slopes and/or broader lateral berms.

A risk to the levee in this reach is posed by submerged granular fill that can build up pore water pressures during an earthquake. In areas where the submerged fill would consist of only a few feet of granular material, such as for the floating levee alternative, it is anticipated that developed pore water pressures and resulting deformations will be small. However, a potential for large deformation exists in the areas where large thicknesses of granular materials will be placed under water, as is the cases for predredging or Bay Mud displacement in open water or where the new levee will adjoin the existing salt pond levee. The potential for deformations in these areas could be reduced by using a relatively clean, coarse granular fill material such that the insitu permeability is high enough to allow rapid dissipation of excess pore water pressures during the period of earthquake shaking. It is anticipated that bank run materials meeting the gradation requirements described in this report would be capable of fairly rapid pore pressure dissipation and would have a low liquefaction susceptibility.

In the vicinity of Boring 1, where Bay Mud is underlain by loose to medium dense silty sand, the stability of the levee may be governed by the presence of the sand layer. Improvement of dike behavior in this area may not be possible unless the sand layer is excavated or insitu techniques are employed to densify the sand layer. Such insitu techniques include vibroflotation and grouting. However, considering the high costs of these techniques and the low probability of strong earthquake shaking occuring simultaneously with high flood levels in the channel, it is doubtful that an analysis of the benefits versus costs would justify the implementation of either vibroflotation or grouting as mitigating measures.

Reach 1B - South Levee Alignment Along Sludge Lagoons

In this reach between the salt pond area and the Milpitas Pumping Station (Borings 8 to 17), the levee alignment passes adjacent to and through the sludge lagoon area. The proposed levee will abut the existing sludge lagoon levees over a substantial portion of this reach.

Subsurface Soil Conditions - The subsurface conditions are illustrated on the profile in Figure 3b. In the westerly part of the reach, closest to the salt pond, the soils consist of soft Bay Mud underlain by stiff, silty clay. In the central part of the reach, stiff, silty clay alluvium apparently predominates. Further east, the shallow alluvium appears to consist predominantly of silt and sand, underlain by stiff clay.

Estimated Levee Behavior - Where the levee alignment is underlain by Bay Mud, displacements of about 3 feet or less are estimated for the case of a maximum earthquake and no consolidation of the existing mud in excess of the present overburden pressure. Displacements would be smaller for a 100-year earthquake. The potential for displacement will decrease with time as the Bay Mud consolidates and gains strength.

Where the levee is underlain by stiff, silty clay alluvium, negligible deformations are expected even during a maximum earthquake. Where the levee is directly underlain by sand and silt alluvium, several feet of levee displacement are possible due to liquefaction of these granular soils.

Mitigating Measures - If it is desired to reduce the potential for levee deformation in areas underlain by Bay Mud, this could be accomplished by using flatter levee side slopes and a broader lateral berm or by excavating the Bay Mud to permit construction of the new levee fill directly on the stronger, underlying clay. Reduction of the potential for large deformations in areas underlain by saturated alluvial sands and silts would require excavation and replacement or insitu densification of these soils as discussed above for Reach 1A.

Reach 1B - North and East Levee Alignments

The north and east levee alignments extend from the salt pond to the Milpitas Sanitation District facility from west of Boring 9 to Boring 16.

Subsurface Soil Conditions - The subsurface soils vary from alluvium to Bay Mud along this reach. Where Bay Mud is present, it is typically overlain by several feet of sandy or clayey alluvium; although it is expected that Bay Mud probably is exposed at the surface of the tidal marsh area at the west end of the alignment. Where Bay Mud is absent the alluvium consists of clays in some locations with interbedded shallow sand layers in other locations.

Estimated Levee Behavior - The behavior of the proposed levee in areas where a shallow sand layer exists, with or without an underlying Bay Mud layer, is expected to be governed by the liquefaction potential of the sand. Liquefaction of the sand could result in levee displacements in excess of 10 feet during a maximum or a 100-year earthquake.

In areas where Bay Mud exists and is overlain by clayey alluvium, or no alluvium, the behavior is expected to be governed by the Bay Mud. Similar to the salt pond levee, the performance of the levee in these areas is expected to be time-dependent; that is, it will be a function of the degree of consolidation and associated strength gain of Bay Mud under the new levee fill. Shortly after construction, a maximum earthquake could induce more than 10 feet of displacement, while a 100-year earthquake could displace the levee by as much as 3 feet. However, when the Bay Mud is fully consolidated (expected in about 5 to 10 years), it is estimated that levee displacement might be about 1 to 2 feet due to a maximum earthquake and less due to a 100-year earthquake.

In areas where the Bay Mud is absent and the alluvium is clayey, negligible displacements are expected during maximum or 100-year earthquakes.

Mitigating Measures - In the areas where the levee is underlain by clayey alluvium and Bay Mud, levee displacements could be reduced by using flatter side slopes and/or stabilizing toe berms. The greatest failure potential for the levee exists where loose, saturated sand is underlain by Bay Mud. Excavation and replacement of the sand deposits would be expected to help reduce the failure potential. However, it would be difficult to excavate the sand or place fill above the soft Bay Mud layer.

Reach 2 - Levees from Milpitas Sanitary District facility to Highway 237

Subsurface Soil Conditions - The subsurface soils in this reach consist of sandy and clayey alluvium. Typically, a saturated sand or silty layer is encountered at a depth of a few to several feet below existing natural grade. Borings indicate that the greatest predominance of cohesionless sands and silts is on the east side of the creek; the sand and silt layer appears to be somewhat less continuous on the west side. In the areas where sand and silt were not encountered in the borings the alluvium consists of clayey soils.

Estimated Levee Behavior - The behavior of the levee is expected to be controlled by the presence or absence of the liquefiable shallow sand layer. Where this layer is present, displacements on the order of 10 feet or more could occur due to a maximum or 100-year earthquake. For portions of the levee alignment underlain by clayey alluvium, where the shallow sand layer is not present, the estimated displacements are negligible during a maximum earthquake or a 100-year earthquake.

Mitigating Measures - The potential for levee displacements due to liquefaction of the shallow sand layer could be greatly reduced by replacing this layer with a key of compacted fill near the creekside toe of the levee. This mitigating measure becomes more difficult where the sand layer becomes thicker and extends deeper below the ground water table. In general, the sand layer appears to be thin (less than 3 feet thick) north of Boring 20 on the east side of the creek. Insitu densification techniques, such as vibroflotation, could be considered where the sand layer is thick; however, such measures would be very costly.

LIMITATIONS

The opinions, conclusions and recommendations contained in this report have been based on information obtained from exploratory borings made at widely separated locations, visual reconnaissance of the project alignment, review of available topographic and historic information, and upon local experience and judgment. Locations of Borings 1 through 7 in the salt pond area were approximated by George S. Nolte and Associates using surveying techniques. The locations of the remaining borings drilled for this investigation were approximately determined by tape measurement from existing site features. Elevations noted on the borings were approximately determined by interpolation between elevation contours or spot elevations shown on available topographic maps. The limits and approximate alignments of old sloughs or creek channels shown on the Site and Exploration Plans were based upon approximate correlations between the conditions shown on 1928 and 1971 aerial photographs of the area and those which were shown on a historic map of the area (Circa 1857). The locations of the borings, the previous slough and channel limits, and the elevation data referenced in this report should be considered accurate only to the degree implied by the method used.

It should be expected that variations in the subsurface conditions will be encountered during construction. In order to permit correlation between the soil data obtained during this investigation and the actual conditions encountered during construction, it is recommended that the Geotechnical Engineer be retained to provide construction review during the earthwork and foundation construction phases of the work.

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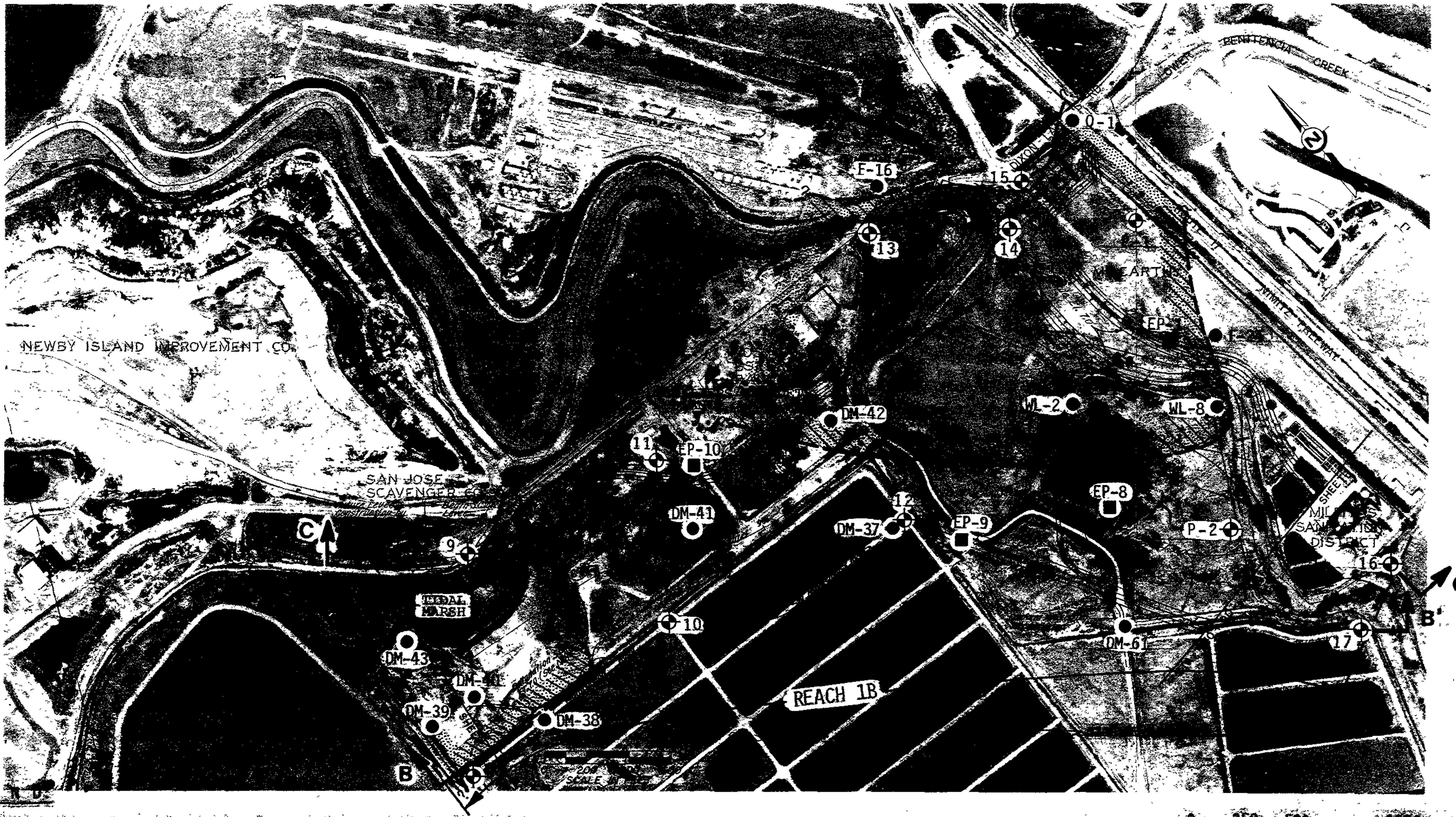


LEGEND

8,10 ⌖ EXPLORATORY BORINGS DRILLED FOR THIS INVESTIGATION

P-1 ⌖ EXPLORATORY BORINGS DRILLED FOR THIS INVESTIGATION IN WHICH SLOTTED PVC STANDPIPE PIEZOMETERS WERE INSTALLED

WL-2 ● EXPLORATORY BORINGS DRILLED PREVIOUSLY BY WOODWARD-LUNDGREN & ASSOCIATES (1972)

DM-37 ● EXPLORATORY BORINGS DRILLED PREVIOUSLY BY DAMES & MOORE (197[illegible])

F-16 ● EXPLORATORY BORINGS DRILLED PREVIOUSLY BY THE U.S. CORPS OF ENGINEERS (1965)

Q-1 ● EXPLORATORY BORINGS DRILLED PREVIOUSLY BY THE CALIFORNIA STATE HIGHWAY DEPT.

EP-1 ■ EXPLORATORY PITS MADE FOR THIS INVESTIGATION

▨ ESTIMATED AREAL EXTENT OF BAY MUD IN REACH 1B

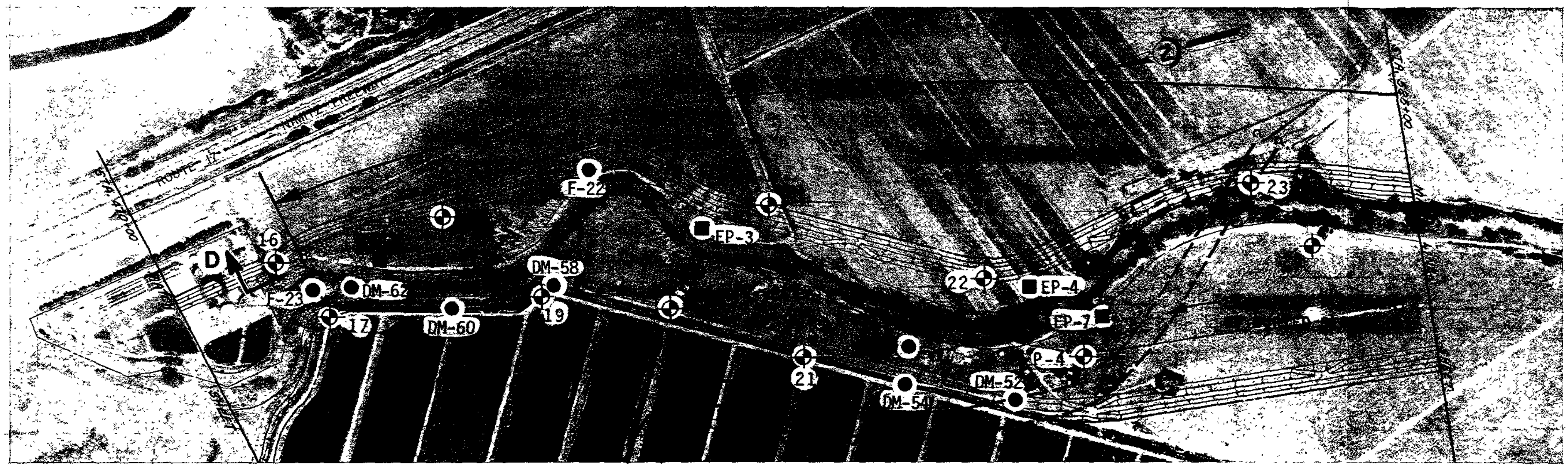
0 250 500 1000 1500
SCALE IN FEET

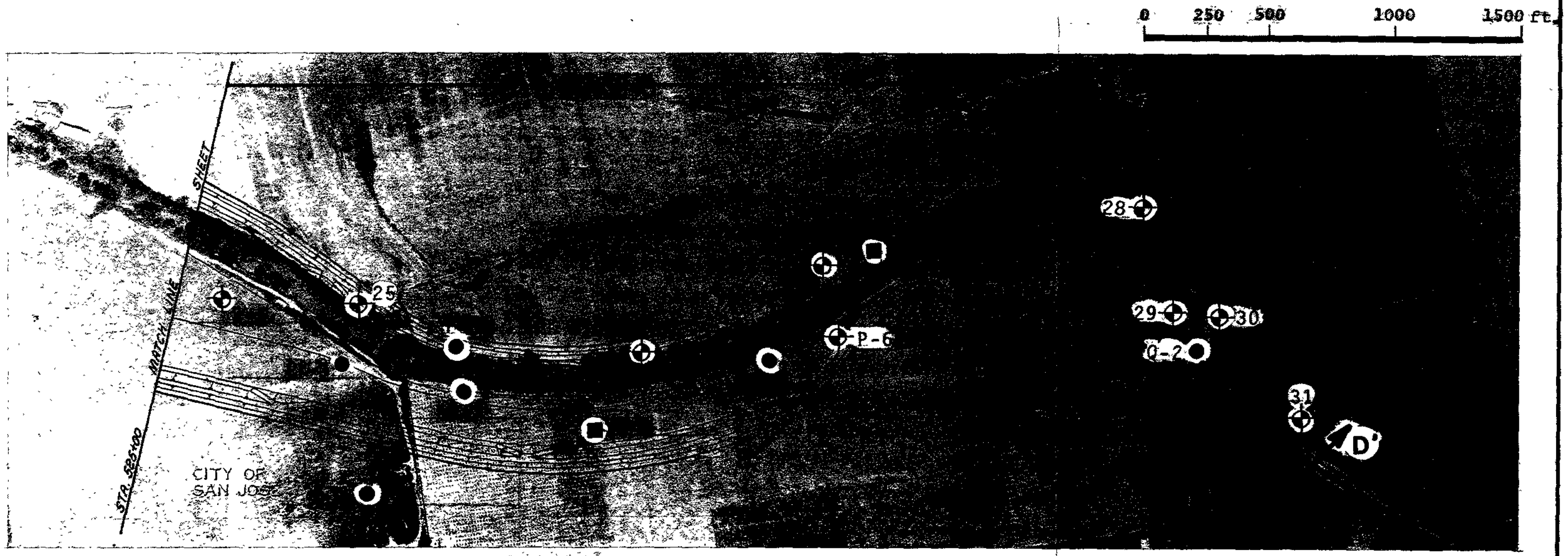
REFERENCE DRAWING: "Coyote Creek", Sheet 6, Prepared by the Santa Clara Valley Water District, Date of Photography: 4-06-83.

SITE AND EXPLORATION PLAN - REACH 1B
COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Project No. 15945H
Woodward-Clyde Consultants

Figure 1B





LEGEND:

16,25	⊕	EXPLORATORY BORINGS DRILLED FOR THIS INVESTIGATION.
P-3	⊕	EXPLORATORY BORINGS DRILLED FOR THIS INVESTIGATION IN WHICH SLOTTED PVC STANDPIPE PIEZOMETERS WERE INSTALLED.
WL-3	●	EXPLORATORY BORINGS DRILLED PREVIOUSLY BY WOODWARD-LUNDGREN & ASSOCIATES (1972).
DM-52	●	EXPLORATORY BORINGS DRILLED PREVIOUSLY BY DAMES & MOORE (1971).
PK-3	●	EXPLORATORY BORINGS DRILLED PREVIOUSLY BY PETER KALDVEER AND ASSOCIATES, INC. (1983).
F-14	●	EXPLORATORY BORINGS DRILLED PREVIOUSLY BY THE U.S. CORPS OF ENGINEERS (1963).
Q-2	●	EXPLORATORY BORINGS DRILLED PREVIOUSLY BY THE CALIFORNIA STATE HIGHWAY DEPARTMENT.
R-16	●	EXPLORATORY BORINGS DRILLED PREVIOUSLY BY COOPER-CLARK & ASSOCIATES.
EP-2	■	EXPLORATORY PITS MADE FOR THIS INVESTIGATION.

REFERENCE DRAWINGS:

"Coyote Creek, Plan and Profile", Sheets 8 and 9, Prepared by the Santa Clara Valley Water District; Date of Photography: 4-06-83.

NOTES:

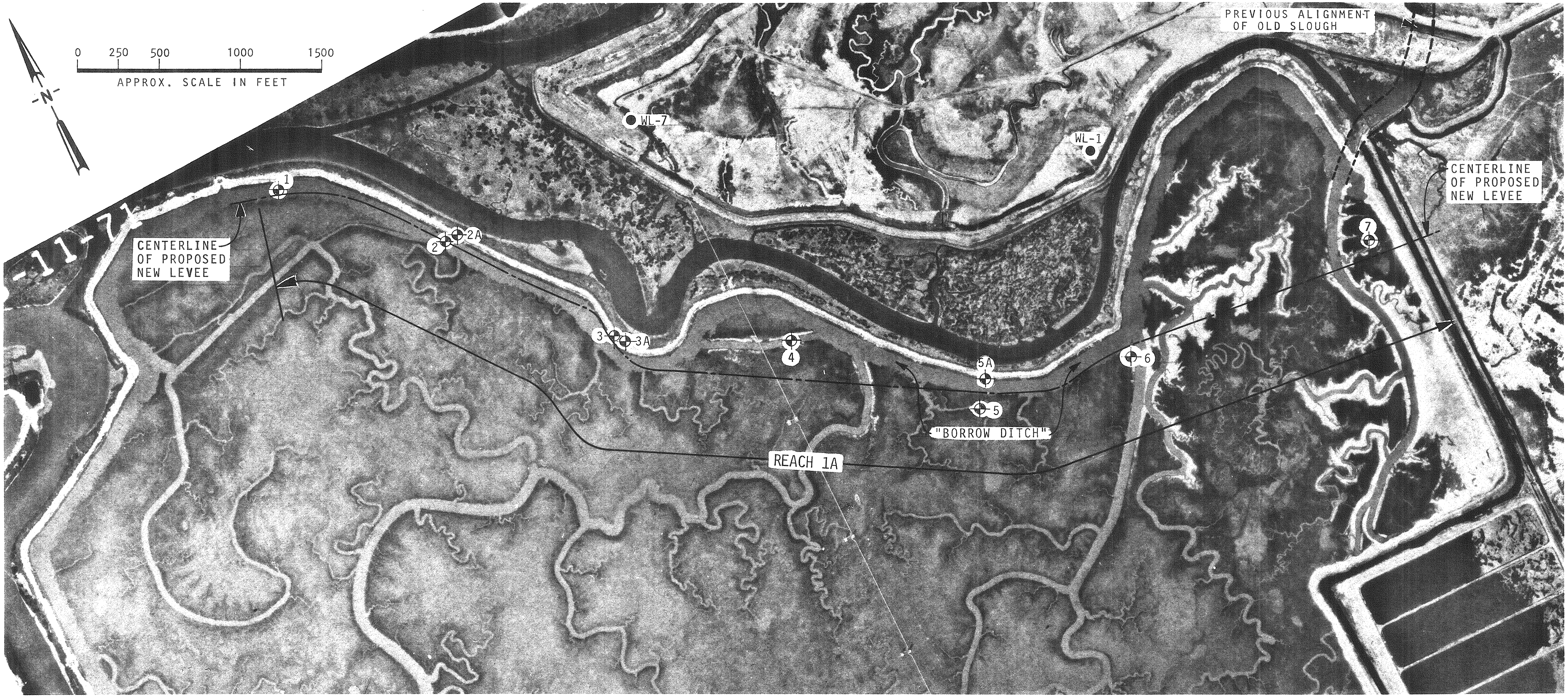
(1) The old buried slough alignment has been based upon tonal differences evident on a 1928 aerial photograph and available subsurface data.

(2) The estimated alignment of the old creek channel has been based upon features shown on "Farm Map Number Two" (mid-1800's) of the Historical Atlas of Santa Clara County.

SITE AND EXPLORATION PLAN - REACH 2
COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Project No. 15945W
Woodward-Clyde Consultants

Figure 1c


0
250
500
1000
1500
APPROX. SCALE IN FEET
N
-11-71
PREVIOUS ALIGNMENT OF OLD SLOUGH
WL-7
WL-1
CENTERLINE OF PROPOSED NEW LEVEE
1
2
2A
CENTERLINE OF PROPOSED NEW LEVEE
7
3
3A
4
5A
6
5
"BORROW DITCH"
REACH 1A

LEGEND:

1,2A ⌖ EXPLORATORY BORINGS DRILLED FOR THIS INVESTIGATION

WL-1 ● EXPLORATORY BORINGS DRILLED PREVIOUSLY BY WOODWARD-LUNDGREN & ASSOCIATES (1972)

REFERENCE:

Pacific Aerial Surveys Photo AVM-813;
Date of Photography: 8-11-71.

NOTE:

The extension of the old slough has been approximated from surface features shown on a 1928 aerial photo in the files of the Leslie Salt Co.

SLOUGH MAP - REACH 1A

COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Project No. 15945W
Woodward-Clyde Consultants

Figure 2

NOTES:

1. See Figures 1a through 1c for section and boring locations.
2. The Idealized Soil Profile has been constructed by direct interpolation between borings drilled at varying spacings and projected to the section line. The dashed lines connecting the various layers at each boring location are for schematic illustration purposes only and should not be construed to represent the actual conditions in the field. Gradational changes shown within the various layers should also be considered approximate.
3. For detailed descriptions of materials encountered in each boring drilled for this investigation, see Figures A-1 through A-40.
4. The approximate pond bottom grade and water surface have been based upon topographic and hydrographic information provided by the Santa Clara Valley Water District.
5. The approximate locations of sloughs have been based upon aerial photo AVM-813 (August 1971), by Pacific Aerial Surveys.

LEGEND:

▽ APPROXIMATE SALT POND WATER SURFACE

0 100 200 400 600
HORIZONTAL SCALE IN FEET

Vertical Exaggeration = 20X

SECTION A-A'
IDEALIZED SOIL PROFILE - REACH 1A
SALT POND AREA
COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Project No. 15945W
Woodward-Clyde Consultants
Figure 3a

B
B'
+20
+10
0
-10
-20
-30
ELEVATION, feet
MATCH LINE SECTION A-A'
BORING #8
BORING #10
BORING #12
BORING #17
SLUDGE LAGOON
TOP OF SLUDGE LAGOON LEVEE
SILTY CLAY FILL (CL-CH), Moderately to well compacted
SILTY CLAY FILL (CL) Moderately to well compacted
SILTY CLAY - CLAYEY SILT FILL (CL-ML) Well compacted
DM-38
DM-36
DM-37
DM-61
BAY MUD (CH)
(1971)
SILTY CLAY (CL-CH) Very stiff
SILTY CLAY (CL) Medium stiff
SANDY SILT (ML) Loose
SILTY CLAY (CH), Stiff
SILTY SAND (SP) Medium dense
SILTY CLAY (CL-ML) Medium
SILTY CLAY (CH) Medium stiff to stiff
SANDY SILT (ML) Loose, grading to (ML-SM), Medium dense
SILTY CLAY (CL) Stiff
SILTY SAND (SM) Medium dense

NOTES:

1. See Figures 1a through 1c for section and boring locations.
2. The Idealized Soil Profile has been constructed by direct interpolation between borings drilled at varying spacings and projected to the section line. The dashed lines connecting the various layers at each boring location are for schematic illustration purposes only and should not be construed to represent the actual conditions in the field. Gradational changes shown within the various layers should also be considered approximate.
3. For detailed descriptions of materials encountered in each boring drilled for this investigation, see Figures A-1 through A-40.
4. The existing grade indicated was taken from topographic information provided by the Santa Clara Valley Water District; date of photography, June 1983.
5. The estimated original ground surface (1971) was taken from topographic information presented on plans entitled "San Jose-Santa Clara, California, Water Pollution Control Plant 1970 Additions", dated May 1971.

LEGEND:

—·—·—	ESTIMATED ORIGINAL GROUND SURFACE (1971)
▨	ESTIMATED LIMITS OF BAY MUD REMOVED DURING SLUDGE LAGOON GRADING
▼	WATER LEVEL RECORDED DURING DRILLING FOR THIS INVESTIGATION
(1971) ▼	WATER LEVEL RECORDED AT TIME OF DRILLING IN 1971

0 100 200 400 600
HORIZONTAL SCALE IN FEET

Vertical Exaggeration = 20X

SECTION B-B'
IDEALIZED SOIL PROFILE - REACH 1B
SOUTH LEVEE ALIGNMENT
COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Project No. 15945W
Woodward-Clyde Consultants
Figure 3b

NOTES:

1. See Figures 1a through 1c for section and boring locations.
2. The Idealized Soil Profile has been constructed by direct interpolation between borings drilled at varying spacings and projected to the section line. The dashed lines connecting the various layers at each boring location are for schematic illustration purposes only and should not be construed to represent the actual conditions in the field. Gradational changes shown within the various layers should also be considered approximate.
3. For detailed descriptions of materials encountered in each boring drilled during this investigation, see Figures A-1 through A-40.
4. The existing grade indicated was taken from topographic information provided by the Santa Clara Valley Water District, June 1983.

LEGEND:

(1971) ▼	WATER LEVEL RECORDED AT TIME OF DRILLING IN 1971.
▼	WATER LEVEL RECORDED DURING DRILLING FOR THIS INVESTIGATION.

0 100 200 400 600
HORIZONTAL SCALE IN FEET

Vertical Exaggeration = 20X

SECTION C-C′
IDEALIZED SOIL PROFILE - REACH 1B
NORTH AND EAST LEVEE ALIGNMENT
COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Project No. 15945W
Woodward-Clyde Consultants

Figure 3c

D
ELEVATION, feet
+20
+10
0
-10
-20
BORING #16
BORING #18
BORING #20
BORING #22
BORING #23
Stiff
SILTY CLAY (CL), Medium to stiff, grading to CLAYEY SILT (ML), soft
SILTY SAND (SM), Loose
SILTY CLAY (CH), Medium to stiff
SILTY CLAY (CL) to (CL-CH), Very stiff
SANDY SILT (ML), Very loose
CLAYEY SILT - SILTY CLAY (ML-CL), Medium to stiff
with lenses of SANDY SILT (ML)
GRAVELLY SAND (SW), Very loose
SILTY SAND (SM), Loose
SILTY CLAY (CH), Medium
Stiff
SILTY CLAY (CL), Very stiff
FILL
FILL
SILTY CLAY - CLAYEY SILT (CL-ML), Soft to medium
SANDY SILT - SILTY SAND (ML-SM), Loose
SILTY CLAY (CL-CH), Medium
BAY MUD (CH) Soft
SILTY SAND (SM), Loose to medium dense
MATCH TO SECTION BELOW
+20
+10
0
-10
-20
ELEVATION, feet
+30
+20
+10
0
-10
-20
MATCH TO SECTION ABOVE
BORING #25
BORING #26
BORING #27
BORING #28
HWY. 237
BORING #30
BORING #31
D'
FILL
FILL
FILL
FILL
SANDY CLAYEY SILT (ML), Medium
SILTY CLAY - CLAYEY SILT (CL-ML), Soft
Loose
Medium dense
SANDY GRAVEL (GP-GM), Medium dense
SILTY SAND (SM) Loose
SANDY CLAY (CL-SC), Medium
SAND (SP), Medium dense with gravel
SILTY CLAY - CLAYEY SILT (CL-ML), Medium
SILTY CLAY (CH), Very stiff grading to (CL)
SANDY SILT - SILTY SAND (ML-SM), Loose
OLD CREEK DEPOSITS
SILTY CLAY (CL), Medium
SILTY SAND (SM), grading to GRAVELLY SAND (SW)
SILTY CLAY - CLAYEY SILT (CL-ML), Stiff with inter-beds of SAND (SM-SP)
SILTY CLAY (CL-CH), Very stiff
+30
+20
+10
0
-10
-20

NOTES:

1. See Figures 1a through 1c for sections and boring locations.
2. The Idealized Soil Profile has been constructed by direct interpolation between borings drilled at varying spacings and projected to the section line. The dashed lines connecting the various layers at each boring location are for schematic illustration purposes only and should not be construed to represent the actual conditions in the field. Gradational changes shown within the various layers should alsa be considered approximate.
3. For detailed descriptions of materials encountered in each boring drilled during this investigation, see Figures A-1 through A-40.
4. The existing grade indicated was taken from topographic information provided by the Santa Clara Valley Water District, dated June 1983.

LEGEND:

▼ WATER LEVEL RECORDED DURING DRILLING FOR THIS INVESTIGATION

0 100 200 400 600
HORIZONTAL SCALE IN FEET

Vertical Exaggeration = 20X

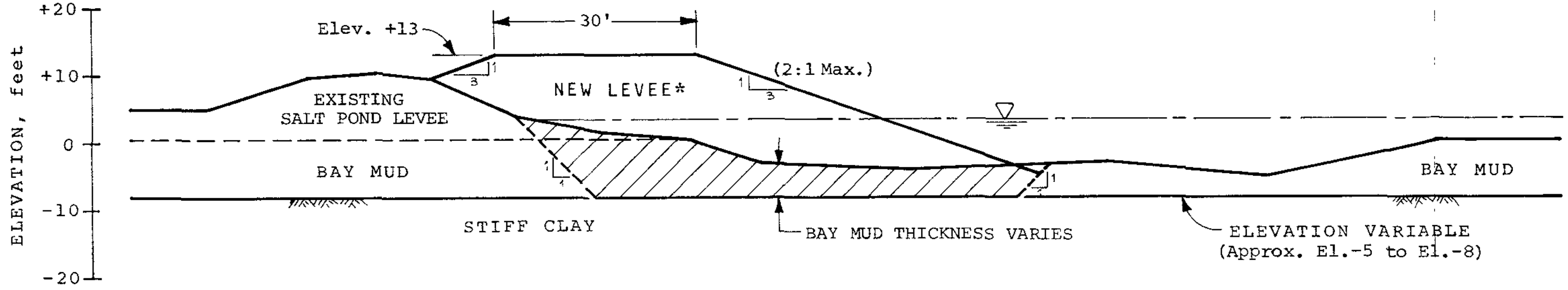
SECTION D-D'
IDEALIZED SOIL PROFILE - REACH 2
MILPITAS SANITATION DIST. TO HWY. 237
COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Project No. 15945W
Woodward-Clyde Consultants

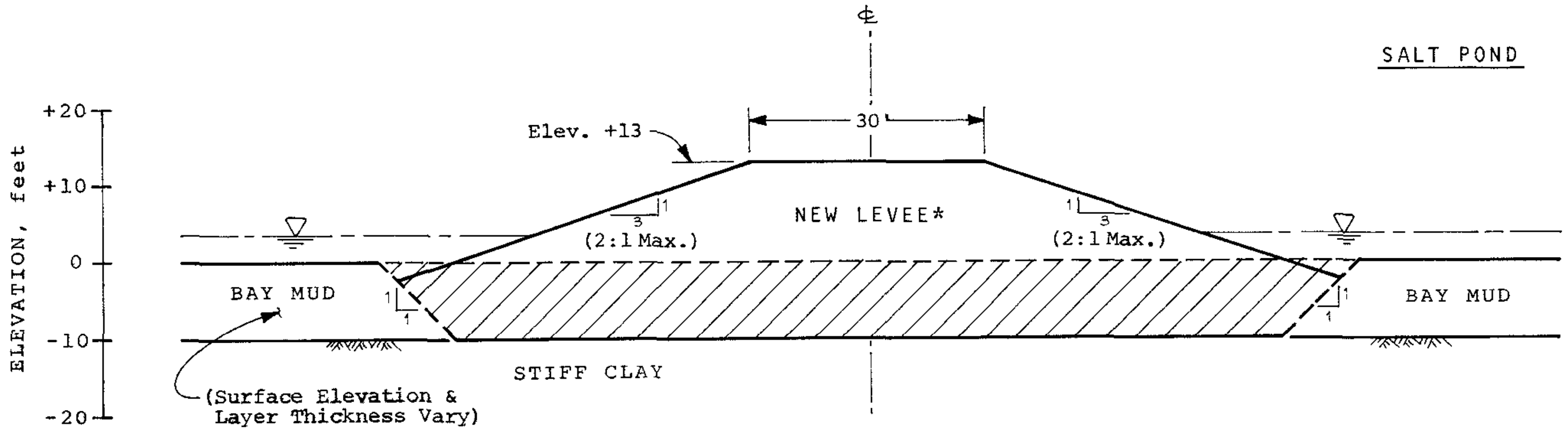
Figure 3d

ADJACENT TO SALT POND LEVEE

(Typical Section at Sta. 79+30)



OPEN WATER



*NOTE:

BELOW WATER PORTION OF NEW LEVEE FILL TO CONSIST OF "BANK RUN" OR "CRUSHER RUN" QUARRY MATERIALS. ABOVE WATER PORTION TO CONSIST OF IMPORT OR ON-SITE COHESIVE EMBANKMENT FILL. BANK RUN MATERIALS MIGHT HAVE PROPERTIES CONSISTENT WITH THOSE OF COHESIVE FILL.

LEGEND:

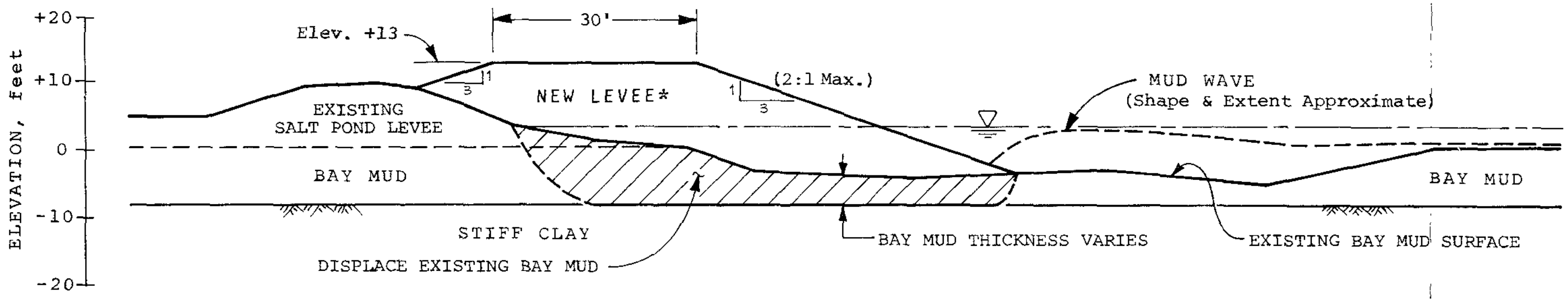
▨ EXISTING BAY MUD TO BE DREDGED

DREDGING ALTERNATIVE: REACH 1A
COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

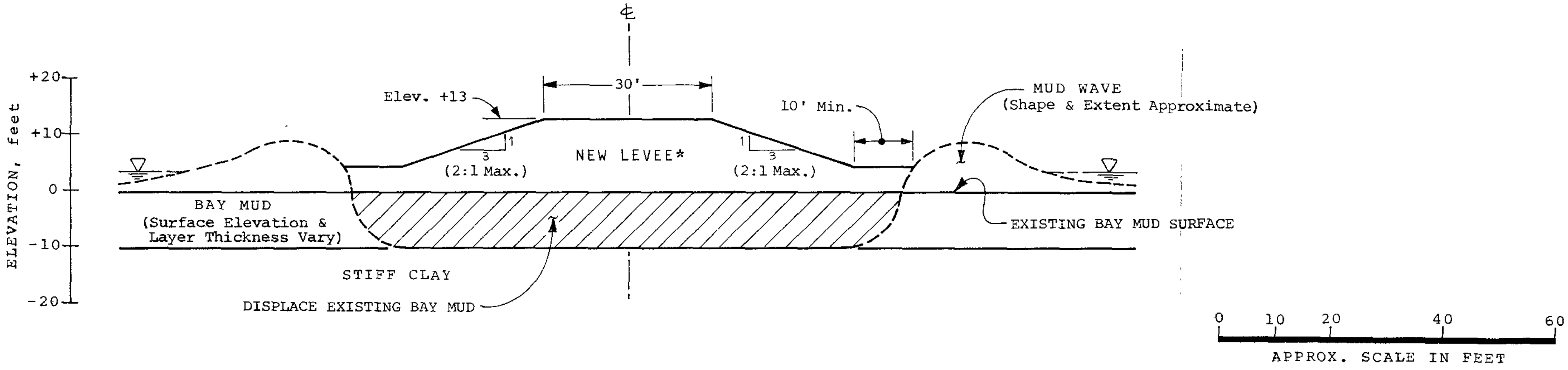
Project No. 15945W
Woodward-Clyde Consultants

Figure 4

ADJACENT TO SALT POND LEVEE
(Typical Section at Sta. 79+30)



OPEN WATER



*NOTE:

BELOW WATER PORTION OF NEW LEVEE FILL TO CONSIST OF "BANK RUN" OR "CRUSHER RUN" QUARRY MATERIALS. ABOVE WATER PORTION TO CONSIST OF IMPORT OR ON-SITE COHESIVE EMBANKMENT FILL. BANK RUN MATERIALS MIGHT HAVE PROPERTIES CONSISTENT WITH THOSE OF COHESIVE FILL.

DISPLACEMENT ALTERNATIVE: REACH 1A

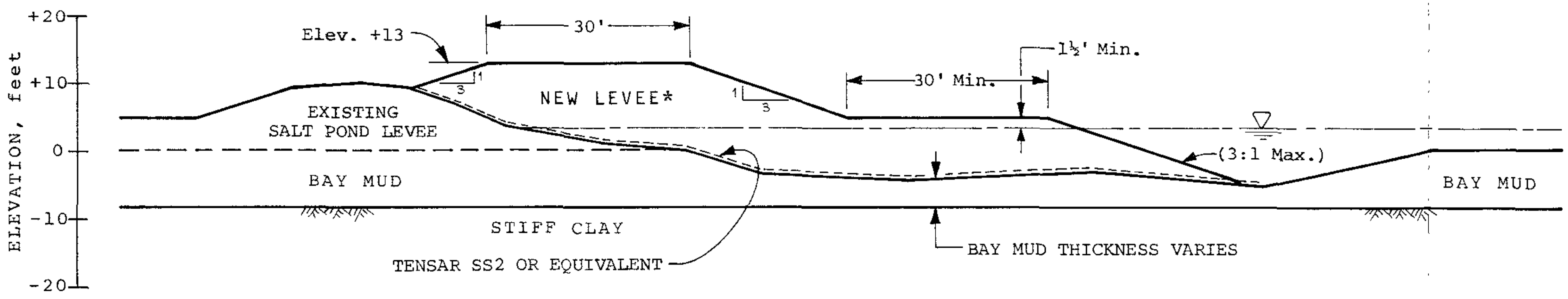
COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Project No. 15945W
Woodward-Clyde Consultants

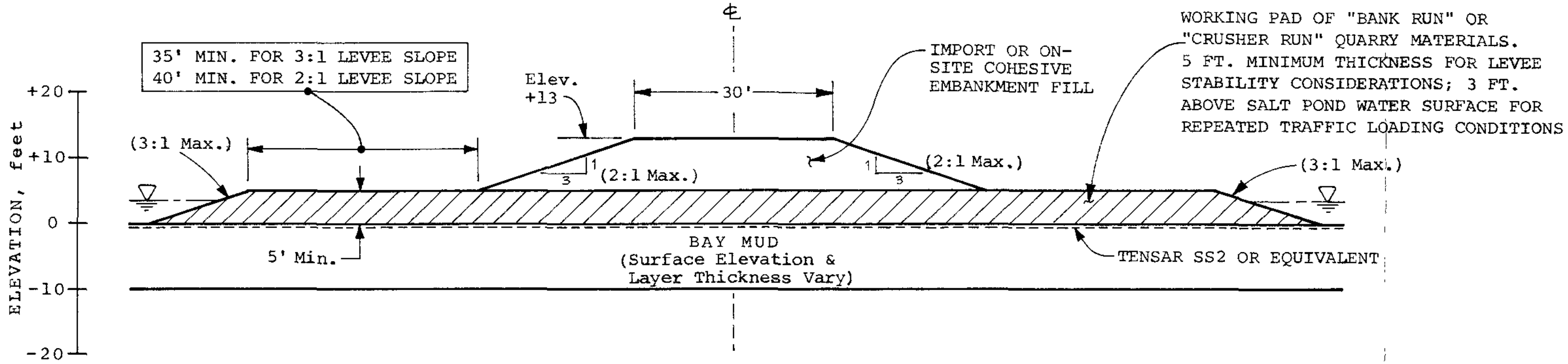
Figure 5

ADJACENT TO SALT POND LEVEE

(Typical Section at Sta. 79+30)



OPEN WATER


℄
35' MIN. FOR 3:1 LEVEE SLOPE
40' MIN. FOR 2:1 LEVEE SLOPE
Elev.
+13
30'
IMPORT OR ON-
SITE COHESIVE
EMBANKMENT FILL
WORKING PAD OF "BANK RUN" OR
"CRUSHER RUN" QUARRY MATERIALS.
5 FT. MINIMUM THICKNESS FOR LEVEE
STABILITY CONSIDERATIONS; 3 FT.
ABOVE SALT POND WATER SURFACE FOR
REPEATED TRAFFIC LOADING CONDITIONS
+20
+10
0
-10
-20
ELEVATION, feet
(3:1 Max.)
1
3
(2:1 Max.)
1
3
(2:1 Max.)
(3:1 Max.)
5' Min.
BAY MUD
(Surface Elevation &
Layer Thickness Vary)
TENSAR SS2 OR EQUIVALENT

*NOTE:

BELOW WATER PORTION OF NEW LEVEE FILL TO CONSIST OF "BANK RUN" OR "CRUSHER RUN" QUARRY MATERIALS. ABOVE WATER PORTION TO CONSIST OF IMPORT OR ON-SITE COHESIVE EMBANKMENT FILL. BANK RUN MATERIALS MIGHT HAVE PROPERTIES CONSISTENT WITH THOSE OF COHESIVE FILL.

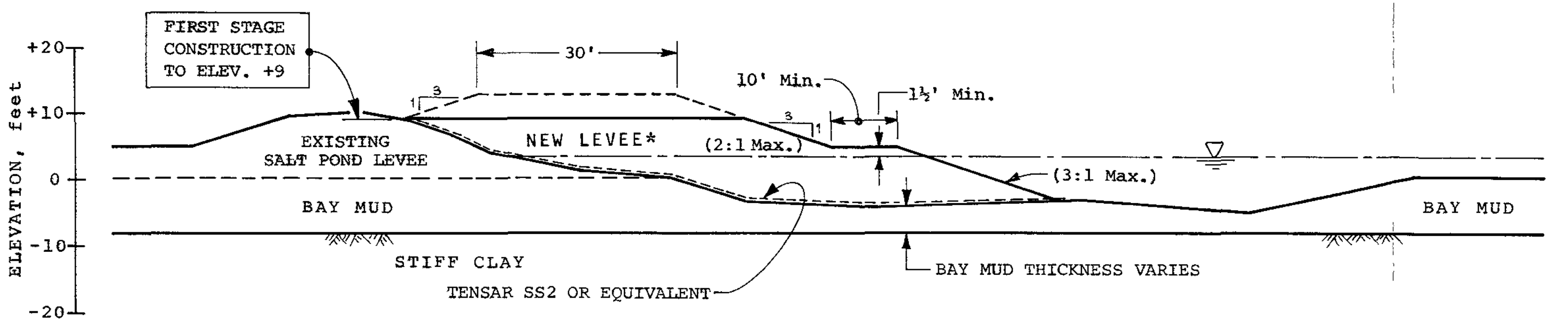
FLOATING EMBANKMENT ALTERNATIVE
(ELEVATION 13): REACH 1A
COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Project No. 15945W
Woodward-Clyde Consultants

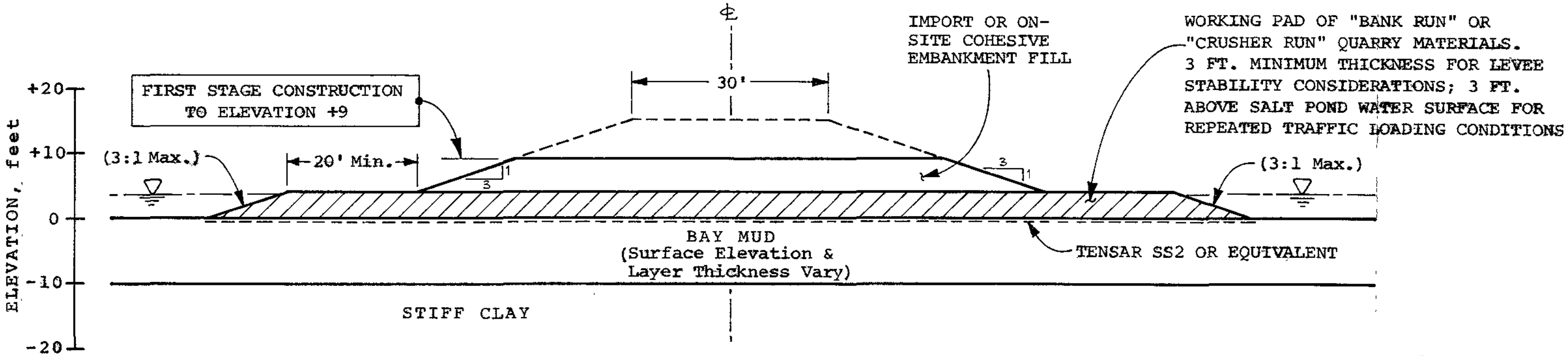
Figure 6

ADJACENT TO SALT POND LEVEE

(Typical Section at Sta. 79+30)



OPEN WATER


℄
30'
IMPORT OR ON-SITE COHESIVE EMBANKMENT FILL
WORKING PAD OF "BANK RUN" OR "CRUSHER RUN" QUARRY MATERIALS. 3 FT. MINIMUM THICKNESS FOR LEVEE STABILITY CONSIDERATIONS; 3 FT. ABOVE SALT POND WATER SURFACE FOR REPEATED TRAFFIC LOADING CONDITIONS
FIRST STAGE CONSTRUCTION TO ELEVATION +9
(3:1 Max.)
20' Min.
3
1
(3:1 Max.)
BAY MUD (Surface Elevation & Layer Thickness Vary)
TENSAR SS2 OR EQUIVALENT
STIFF CLAY
ELEVATION, feet
+20
+10
0
-10
-20

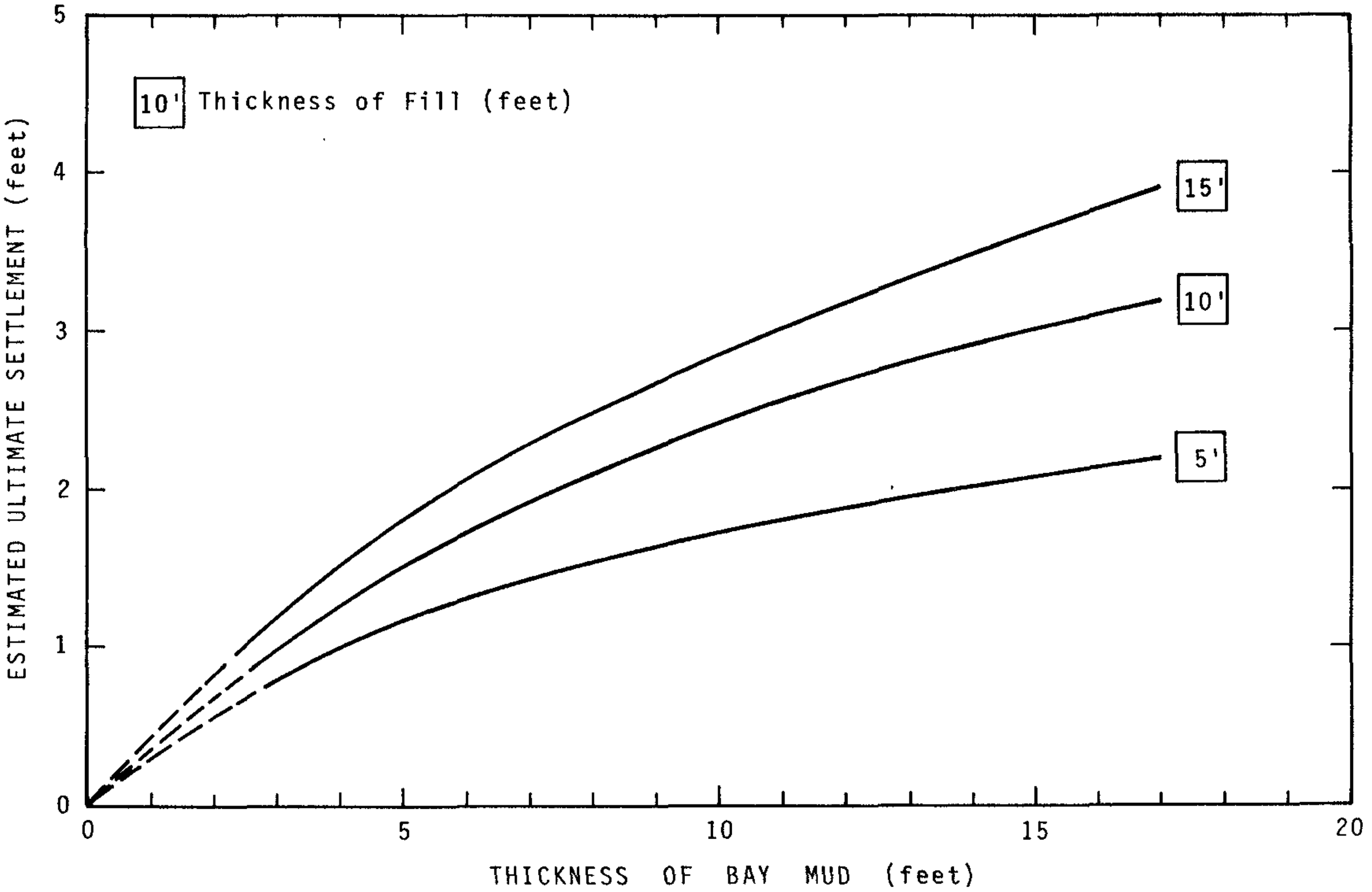
*NOTE:

BELOW WATER PORTION OF NEW LEVEE FILL TO CONSIST OF "BANK RUN" OR "CRUSHER RUN" QUARRY MATERIALS. ABOVE WATER PORTION TO CONSIST OF IMPORT OR ON-SITE COHESIVE EMBANKMENT FILL. BANK RUN MATERIALS MIGHT HAVE PROPERTIES CONSISTENT WITH THOSE OF COHESIVE FILL.

FLOATING EMBANKMENT ALTERNATIVE
STAGE CONSTRUCTION: REACH 1A
COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Project No. 15945W
Woodward-Clyde Consultants

Figure 7

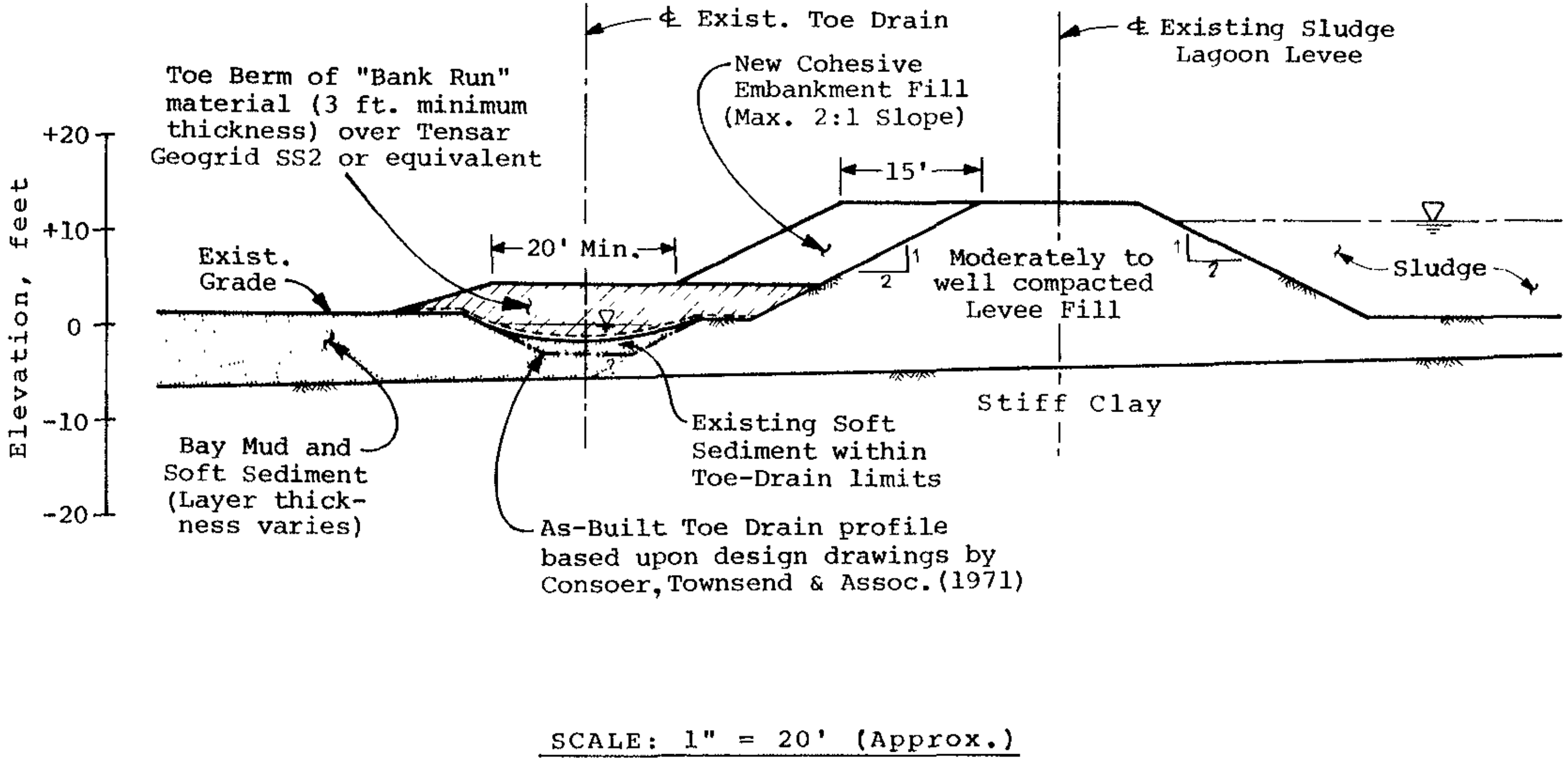


ESTIMATED ULTIMATE SETTLEMENT
FLOATING EMBANKMENT ALTERNATIVE
COYOTE CREEK IMPROVEMENTS

Figure 8

Project No. 15945W

Woodward-Clyde Consultants


℄ Exist. Toe Drain
℄ Existing Sludge Lagoon Levee
Toe Berm of "Bank Run" material (3 ft. minimum thickness) over Tensar Geogrid SS2 or equivalent
New Cohesive Embankment Fill (Max. 2:1 Slope)
15'
20' Min.
+20
+10
0
-10
-20
Elevation, feet
Exist. Grade
1
2
Moderately to well compacted Levee Fill
1
2
Sludge
Stiff Clay
?
Bay Mud and Soft Sediment (Layer thickness varies)
Existing Soft Sediment within Toe-Drain limits
As-Built Toe Drain profile based upon design drawings by Consoer, Townsend & Assoc. (1971)
SCALE: 1" = 20' (Approx.)

Project No. 15945W

Woodward-Clyde Consultants

NEW LEVEE ADJACENT TO SLUDGE LAGOON
(STA. 70 TO 80) - LATERAL BERM ALTERNATIVE
COYOTE CREEK IMPROVEMENTS

Figure 9

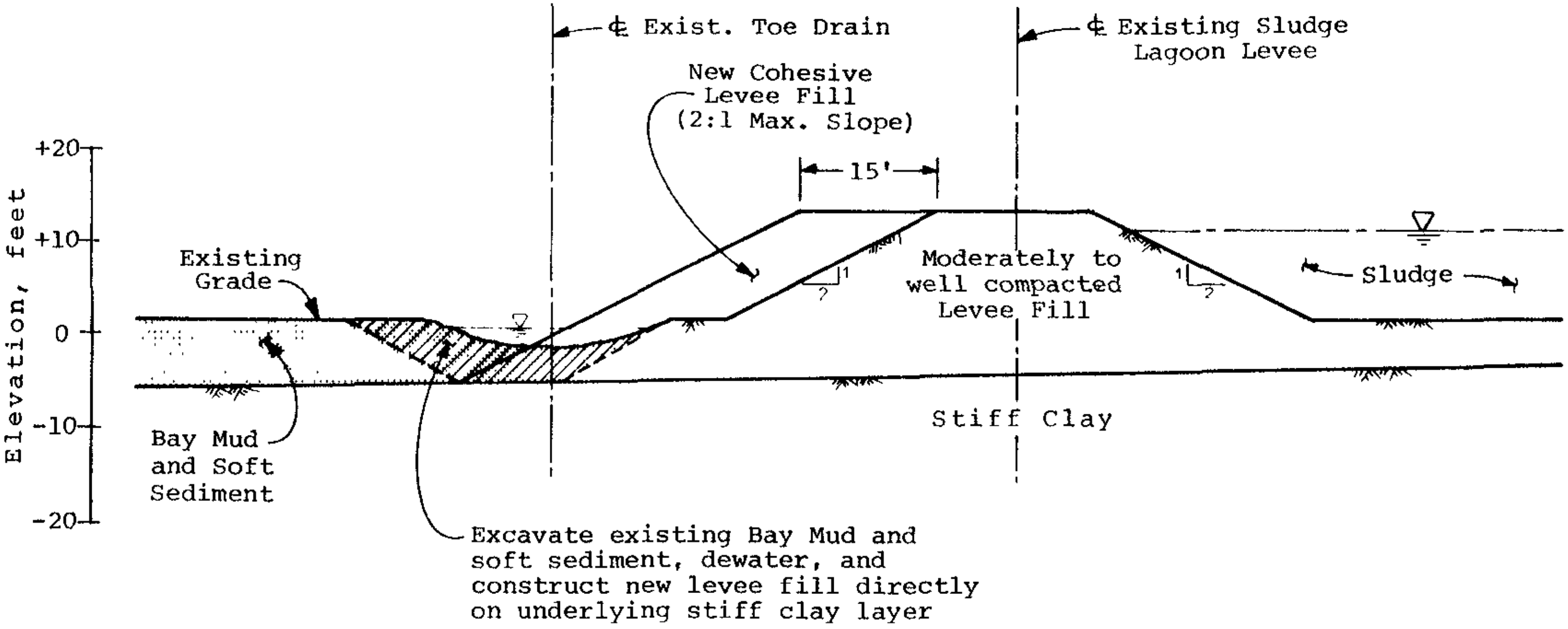
Project No. 15945W

Woodward-Clyde Consultants

NEW LEVEE ADJACENT TO SLUDGE LAGOON
(STA. 70 TO 80) - EXCAVATION ALTERNATIVE

COYOTE CREEK IMPROVEMENTS

Figure 10



℄ Exist. Toe Drain
℄ Existing Sludge Lagoon Levee
New Cohesive Levee Fill (2:1 Max. Slope)
15'
+20
+10
0
-10
-20
Elevation, feet
Existing Grade
Moderately to well compacted Levee Fill
1
2
Sludge
Stiff Clay
Bay Mud and Soft Sediment
Excavate existing Bay Mud and soft sediment, dewater, and construct new levee fill directly on underlying stiff clay layer

SCALE: 1" = 20' (Approx.)

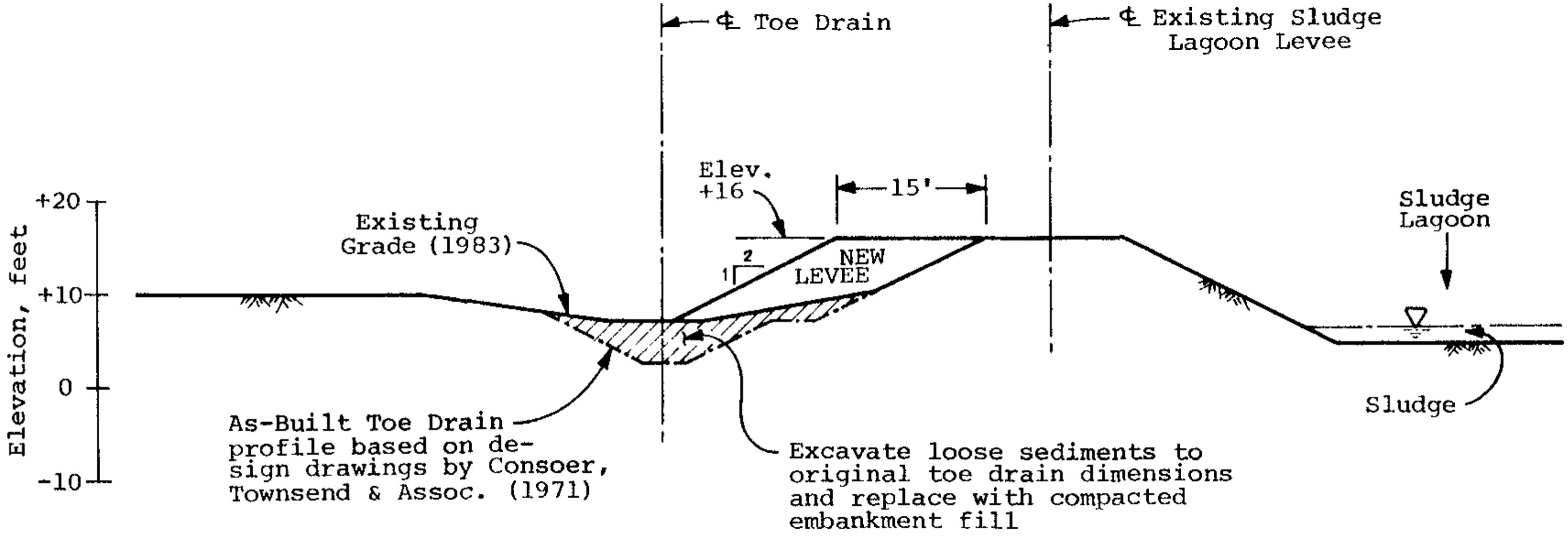
Woodward-Clyde Consultants

Project No. 15945W

NEW LEVEE ADJACENT TO SLUDGE LAGOON (E. OF STA. 80) - OVEREXCAVATION OF TOE DRAIN

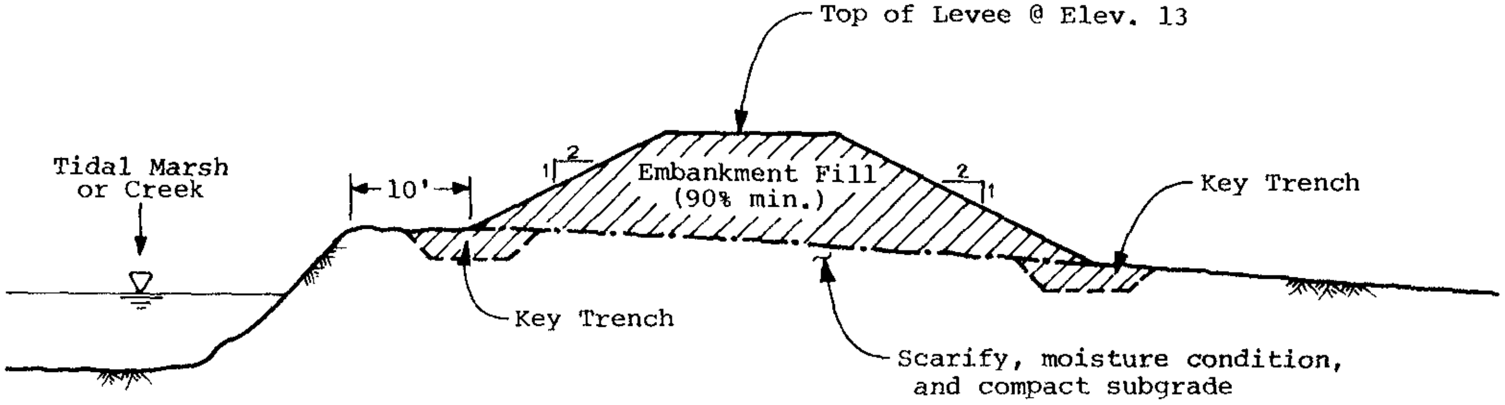
COYOTE CREEK IMPROVEMENTS

Figure 11



℄ Toe Drain
℄ Existing Sludge Lagoon Levee
Elev. +16
15'
+20
+10
0
-10
Elevation, feet
Existing Grade (1983)
2
1
NEW LEVEE
Sludge Lagoon
Sludge
As-Built Toe Drain profile based on design drawings by Consoer, Townsend & Assoc. (1971)
Excavate loose sediments to original toe drain dimensions and replace with compacted embankment fill

SCALE: 1" = 20' (Approx.)


Top of Levee @ Elev. 13
Tidal Marsh or Creek
10'
2
1
Embankment Fill (90% min.)
2
1
Key Trench
Key Trench
Scarify, moisture condition, and compact subgrade

NOTE: Key Trenches should be equipment width (8 feet minimum) and should extend at least 3 feet below existing grade.

SCALE: 1" = 20' (Approx.)

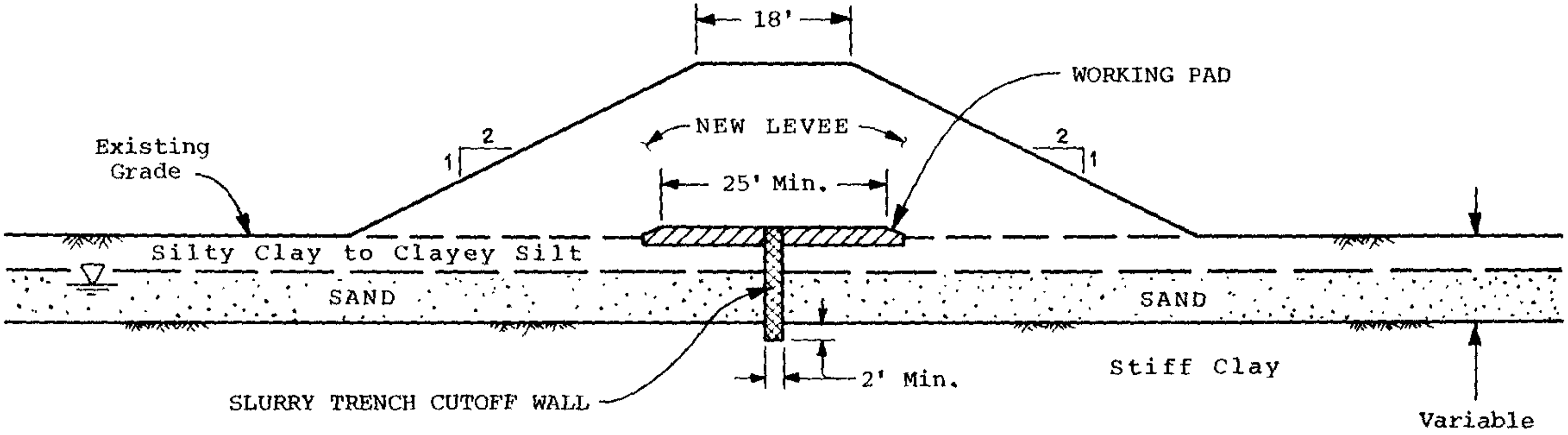
Woodward-Clyde Consultants

Project No. 15945W

KEY TRENCH DETAILS

COYOTE CREEK IMPROVEMENTS

Figure 12



NOTE (1): A working pad at least 25 feet wide should be constructed prior to the start of the trench excavation. The native soils in the pad area should first be scarified, moisture conditioned, and recompacted. Cohesive embankment fill then should be placed and compacted to provide a finished pad surface at least one foot above the adjacent natural grade. A minimum degree of compaction of 90% should be attained.

NOTE (2): The slurry trench excavation should be at least two feet wide and extend at least two feet into the underlying stiff clay.

SCALE: 1" = 20' (Approx.)

Woodward-Clyde Consultants
Project No. 15945W

SLURRY TRENCH CUTOFF DETAILS
COYOTE CREEK IMPROVEMENTS

Figure

APPENDIX A

FIELD INVESTIGATION AND LABORATORY TESTS

COYOTE CREEK IMPROVEMENTS

Santa Clara County, California

FIELD INVESTIGATION

Exploratory Borings

Thirty-four (34) borings were drilled during the course of this investigation to explore the subsurface conditions along the project alignment. Borings 1 through 31 were drilled between the dates of September 16 and October 6, 1983, and Borings 2A, 3A and 5A were drilled on December 7 and 16, 1983. In addition, Borings P-1 through P-6 were drilled on July 2, 1983, as an advance phase of exploration to provide information regarding the depth to groundwater. These six (6) previous borings were fitted with 2-inch diameter slotted PVC standpipe piezometers to permit periodic measurements of the groundwater levels. The approximate locations of all of these borings are shown on the attached Site and Exploration Plans, Figures 1a through 1e. The borings were advanced by either a truck mounted 6-inch continuous flight auger, barge mounted portable wash boring equipment, or by hand auger methods. The methods used to advance the borings are noted on the Logs of Borings, Figures A-1 through A-40.

All of the borings were drilled under the supervision of representatives of Woodward-Clyde Consultants. Visual classifications of the soils encountered were made from the cuttings at the time of drilling. Samples of the underlying soils were obtained using the samplers summarized on the Exploration Legend Sheet attached to this Appendix. The samplers were either driven into the soil or advanced by pushing under hydraulic pressure. Where a free falling hammer was used to drive the sampler into the soils, the number of blows to penetrate the last 12 inches is shown at the corresponding locations on the Logs of Borings. When the samplers were withdrawn from the boring, the samples were carefully removed, sealed to preserve the natural moisture content and returned to the laboratory for testing. Field classifications of the soil samples were verified by further examination and testing in the laboratory.

Exploratory Pits

Ten (10) exploratory backhoe pits also were excavated on December 5, 1983, to supplement the exploratory boring information as well as to obtain bulk soil samples. The approximate locations of the pits are shown on Figures 1c through 1e. Logs of the Exploratory Pits 1 through 9 are presented as Figures A-41 through A-49.

LABORATORY TESTS

The water content, dry density and unconfined compressive strength were determined for selected samples to evaluate the strength and compressibility of the underlying soils. The results of these tests together with the resistance to penetration of the sampler are shown at the corresponding sample locations on the Logs of Borings.

Unconsolidated undrained triaxial compression tests were performed on 4 selected samples of Bay Mud recovered from the salt pond area to estimate the undrained shear strength. The results of these tests are presented graphically on Figures A-50 and A-51.

A consolidation test was made on a selected sample of Bay Mud to assist in settlement analysis. The results of this test are presented graphically on Figure A-52.

Grain size distributions were determined for selected native soil and quarry soil-rock samples. The results of these tests are presented graphically on Figures A-53 through A-62.

Atterberg Limits (liquid and plastic limits) were determined for fine grained soil samples. The results of these tests along with grain size information for other selected samples are presented on Figures A-63 through A-66.

Laboratory compaction tests were performed using bulk samples of the on-site soils obtained from the exploratory pits and selected "bank run" quarry materials. The results of these tests are summarized on Table A-1.

Variable head permeability tests were performed on selected drive samples of the native soils recovered during the exploratory drilling, as well as compacted native and quarry soil samples. The results of these tests are summarized on Table A-2.

FIELD VANE SHEAR TESTS

During the drilling of the exploratory borings in the salt pond area (Reach 1A), field vane shear tests were performed to help estimate the undrained shear strength of the Bay Mud deposits. These insitu tests were performed using a 4 blade rectangular shaped vane, 3 inches in diameter and 6 inches long. After the bore hole had been advanced to the desired depth, the vane was carefully pushed into the soil and the torque necessary to shear the cylinder of soil defined by the vane blades was measured. The vane then was continually rotated until a minimum shear resistance was achieved. Peak shear strength and remolded shear strength values then were computed for the respective torque measurements. Those computed shear strengths are summarized in Table A-3 and the peak shear strength values are presented at the corresponding test depths on the Logs of Borings 1 through 7, 2A, 3A and 5A (Figures A-1 through A-10).

QUARRY SURVEY

In order to investigate potential import borrow sources, a geologic reconnaissance was made of 5 local quarries within a 10 mile radius of the project. A summary of the quarry data is presented on Table A-4.

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

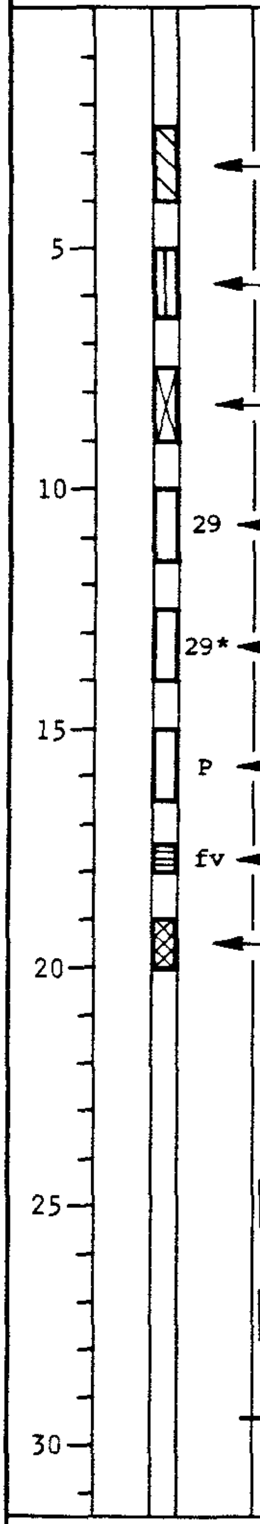
EXPLORATION LEGEND SHEET

Date Drilled: ______________________
Type of Boring: ______________________
Hammer Weight: ______________________
Remarks: ______________________

Depth, Ft	Samples	Blows/Ft	DESCRIPTION	LABORATORY TESTS: Moisture Content, %	LABORATORY TESTS: Dry Density, pcf	LABORATORY TESTS: Unconfined Compressive Strength, psf

Surface Elevation: 3± feet

Based on topographic information provided by the Santa Clara Valley Water District



2-INCH I.D. MODIFIED CALIFORNIA SAMPLER

2-INCH O.D. STANDARD SPLIT-SPOON SAMPLER

3-INCH O.D. SHELBY TUBE SAMPLER

29 ← BLOW COUNT WITH A 140-LB. HAMMER FALLING 30 INCHES

29* ← BLOW COUNT WITH A 50-LB. HAMMER FALLING 30 INCHES

P ← SAMPLER ADVANCED BY HYDRAULIC PUSHING

fv ← FIELD VANE SHEAR TEST

BULK SAMPLE FROM DEPTH INTERVAL INDICATED

+ #4 = 30%
-#200 = 15% ← PERCENTAGE OF MATERIAL RETAINED (+) OR PASSING (-), SIEVE SIZE INDICATED

PI = 30 ← PLASTICITY INDEX

Su(fv)=300psf ← UNDRAINED SHEAR STRENGTH ESTIMATED BY FIELD VANE SHEAR TEST

Su(uu)=300psf ← UNDRAINED SHEAR STRENGTH ESTIMATED BY UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

▽ WATER LEVEL MEASURED:
ATD ← At Time of Drilling
3 Hrs. ← In Hours or Days After Drilling
9-20-83 ← On Date Indicated

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 1

Date Drilled: Sept. 30, & Oct. 1, 1983
Type of Boring: 4-inch wash
Hammer Weight: 50 lbs.

Remarks: Exploratory boring advanced using barge-mounted portable drilling equipment

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 2½± feet

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION		LABORATORY TESTS Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	P	SILTY CLAY (CH) Very soft, dark gray, with high organic material content in top 2½ feet		101	45	100
		fv		Su(fv)=170psf			
5		fv		Su(fv)=210psf			
			(BAY MUD)				
	2	P			45	-	-
		8*	SILTY SAND (SM) Loose, black	-#200 = 22%			
10			With thin interbedded layers of Bay Mud and shells				
	3	14*		-#200 = 19%	-	-	-
15	4	24*	Medium dense		-	-	-
	5	30*			-	-	-
20	6	43*			-	-	-
			SILTY CLAY (CL-CH) Very stiff, gray-brown				
	7	20*	Grades to Sandy Clay (CL), stiff, brown		-	-	-
25			SILTY CLAY (CL) Very stiff, light brown, with lenses of shells				
	8	40*			-	-	-
	9	40*	BOTTOM OF BORING @ 30 feet		-	-	-
30							
			• Salt pond water depth @ boring location = 1± ft.				

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Figure A-1

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 2

Date Drilled: September 29 & 30, 1983
Type of Boring: 4-inch wash
Hammer Weight: 50 lbs.

Remarks: Exploratory boring advanced using barge-mounted portable drilling equipment
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 0± feet

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content,%	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	P	SILTY CLAY (CH) Very soft, dark gray (BAY MUD)	77	54	150
		fv	Su(fv)=90psf			
5		fv	Su(fv)=150psf			
	2	17*	SILTY CLAY (CH) Stiff, dark gray	29	91	2100
10	3	35*	Grading to Silty Clay (CL-CH), stiff to very stiff, gray-brown	22	105	3800
			SILTY CLAY (CL) Very stiff, brown and light gray mottled			
15	4	25* 6"		-	-	-
20						
25						
30						

BOTTOM OF BORING @ 15½ feet

- Salt pond water depth @ boring location = 3½± feet
- 3± inch salt crust covering pond bottom

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Figure A-2

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 2A

Date Drilled: December 7 & 16, 1983
Type of Boring: 4-inch Hand Auger
Hammer Weight: --

Remarks: Boring drilled on top of salt pond levee

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 9± feet

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	FIELD VANE SHEAR TEST RESULTS Su (fv), psf
	▤	fv	SILTY CLAY (CH) Stiff, moist, dark gray, with shrinkage cracks to 4-in. wide and 18-in. deep (DREDGED BAY MUD FILL)	1490
	▤	fv	Medium, dark brown-gray	590
5				
	▤	fv	Wet, gray, with peat	670
	▤	fv	(DREDGED FILL)	450
10	▤	fv	SILTY CLAY (CH)	510
			Soft to medium, gray (BAY MUD)	
	▤	fv		480
15				
	▤	fv		450
	▤	fv	Medium	650
			BOTTOM OF BORING @ 16½ feet	
			● Boring terminated in stiff clay encountered below Bay Mud layer	
20				
25				
30				

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Figure A-3

Project: COYOTE CREEK IMPROVEMENTS Santa Clara County, California	# Log of Boring No. 3

Date Drilled: October 3, 1983
Type of Boring: 4-inch wash
Hammer Weight: 50 lbs.

Remarks: Exploratory boring advanced using barge-mounted portable drilling equipment
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 2½± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moi[illegible] Conte[illegible] %	Dry Density, pcf	Unconfined Compressive Strength, psf
			SILTY CLAY (CH)			
	1	P	Very soft, dark gray, with high organic material content in top 1½ feet	70	57	240
		fv	Su(fv)=370psf			
5	2	P 3*	(BAY MUD)	61	62	440
		fv	Su(fv)=300psf			
10	3	P 11*		43	76	-
			SILTY CLAY (CH) Medium, dark gray			
	4	25*		31	90	1750
			(CL-CH), dark gray-brown			
15						
			SILTY CLAY (CL) Very stiff, brown and light gray mottled			
	5	60*/9"		-	-	-
	6	30*/6"		-	-	-
			BOTTOM OF BORING @ 18½ feet			
20						
25						
30						

- Salt pond water depth @ boring location = 3/4± ft.
- 1± inch salt crust covering pond bottom

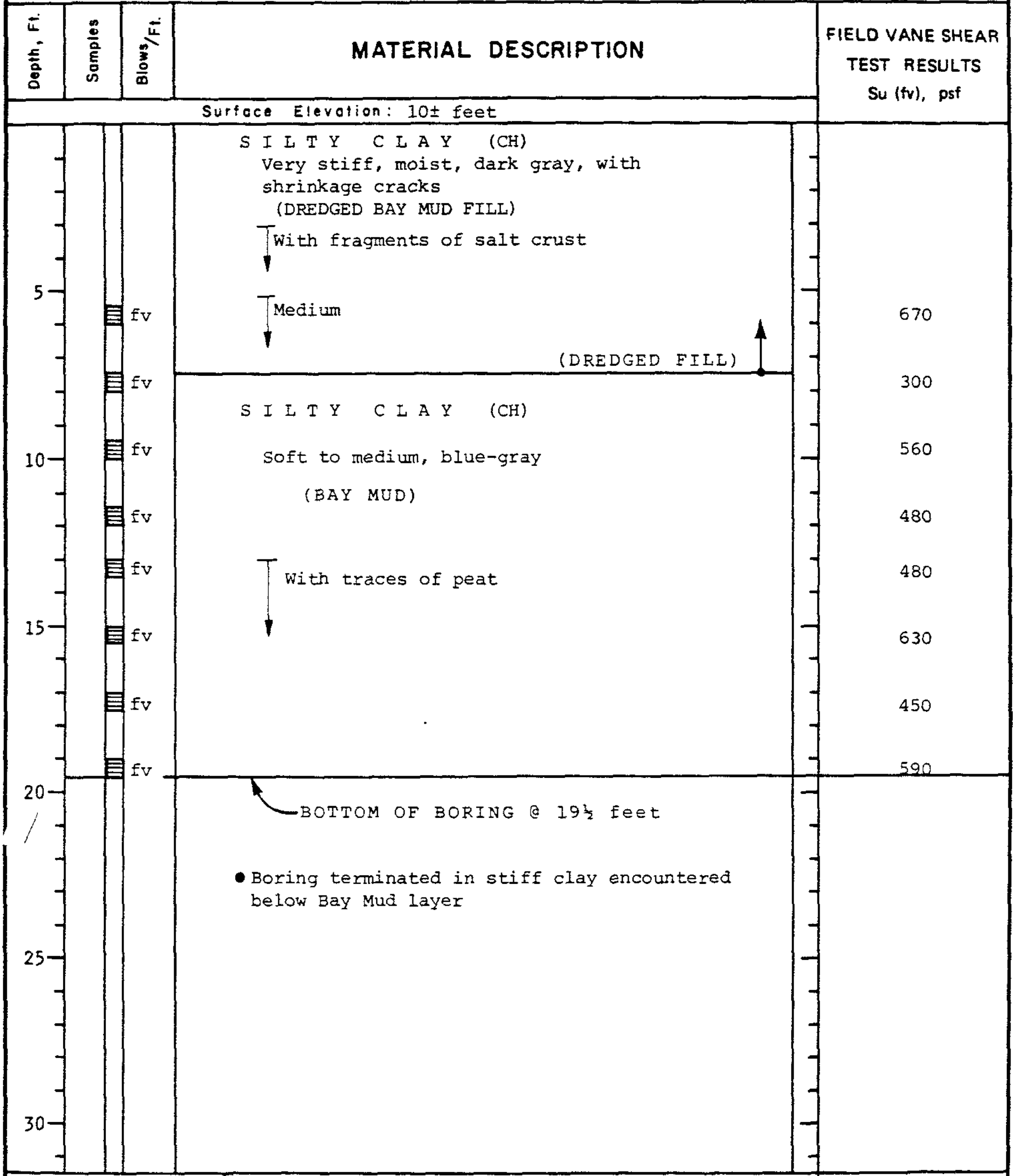
Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 3A

Date Drilled: December 7, 1983
Type of Boring: 4-inch Hand Auger
Hammer Weight: --

Remarks: Boring drilled on top of salt pond levee

(See Legend Sheet for sampler types and hammer weights)



MATERIAL DESCRIPTION

Surface Elevation: 10± feet

SILTY CLAY (CH)
Very stiff, moist, dark gray, with shrinkage cracks
(DREDGED BAY MUD FILL)

With fragments of salt crust

Medium

(DREDGED FILL)

SILTY CLAY (CH)

Soft to medium, blue-gray

(BAY MUD)

With traces of peat

BOTTOM OF BORING @ 19½ feet

- Boring terminated in stiff clay encountered below Bay Mud layer

Blows/Ft.	FIELD VANE SHEAR TEST RESULTS Su (fv), psf
fv	670
fv	300
fv	560
fv	480
fv	480
fv	630
fv	450
fv	590

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Figure A-5

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 4

Date Drilled: October 3, 1983
Type of Boring: 4-inch wash
Hammer Weight: 50 lbs.

Remarks: Exploratory boring advanced using barge-mounted portable drilling equipment
(See Legend Sheet for sampler types and hammer weights)

Depth, Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 2½± feet			
			SILTY CLAY (CH)			
	1	P	Very soft, dark gray, with organic material	62	61	320
		fv	Su(fv)=240psf			
			(BAY MUD)			
5		fv	Su(fv)=340psf			
	2	P 2* 8* 10* 15*	Grading to Clayey Silt (ML), with some fine sand and shell fragments PI = 6	39	81	240
10			SILTY CLAY (CL-CH) Medium, gray-brown	28	-	-
	3	65*	SILTY CLAY (CL) Stiff, brown, with some sand	21	102	3310
15			Very stiff, brown and gray			
	4	60*		20	108	4530
			BOTTOM OF BORING @ 17 feet			
20						
25						
30						

BOTTOM OF BORING @ 17 feet

- Salt pond water depth @ boring location = 1± ft.
- 1± inch salt crust covering pond bottom

Proj. No. 15945W

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Figure A-6

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 5

Date Drilled: October 4, 1983
Type of Boring: 4-inch wash
Hammer Weight: 50 lbs.

Remarks: Exploratory boring advanced using barge-mounted portable drilling equipment

(See Legend Sheet for sampler types and hammer weights)

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	LABORATORY TESTS Dry Density, pcf	LABORATORY TESTS Unconfined Compressive Strength, psf
			Surface Elevation: 2½± feet			
			SILTY CLAY (CH)			
	1	P	Very soft, dark gray, with high organic material content	60	62	240
		fv	Su(fv)=190psf			
5			(BAY MUD)			
	2	P		72	55	240
		fv	Su(fv)=210psf			
10			SILTY CLAY (CH) Stiff, gray			
	3	44*	SILTY CLAY (CL-CH) Very stiff, gray-brown	24 24	101 102	3800 5360
15			SILTY CLAY (CL) Very stiff, brown and light gray mottled			
	4	42*		21	106	4720
20			BOTTOM OF BORING @ 17 feet			
25						
30						

- Salt pond water depth @ boring location = 1± ft.
- 1± inch salt crust covering pond bottom

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Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 5A

Date Drilled: December 16, 1983
Type of Boring: 4-inch Hand Auger
Hammer Weight: --
Remarks: Boring drilled on top of salt pond levee
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 9± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	FIELD VANE SHEAR TEST RESULTS Su (fv), psf
			SILTY CLAY (CH): Stiff, wet, dark gray, with shrinkage cracks (DREDGED BAY MUD FILL)	
			← Salt crust	
			} Organic material (Peat)	
			↓ Gray-brown	
	▤	fv		890
5			↓ Medium	
	▤	fv	ATD (DREDGED FILL) ↑	390
	▤	fv	SILTY CLAY (CH)	490
			Soft, dark gray	
10	▤	fv	(BAY MUD)	330
	▤	fv		450
			↓ Gray-brown, with organic material	
15	▤	fv	↓ Dark gray	330
	▤	fv		420
			BOTTOM OF BORING @ 17½ feet	
20				
25				
30				

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Figure A-8

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 6

Date Drilled: October 5, 1983

Type of Boring: 4-inch wash

Hammer Weight: 50 lbs.

Remarks: Exploratory boring advanced using barge-mounted portable drilling equipment

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 1½± feet

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	LABORATORY TESTS Dry Density, pcf	LABORATORY TESTS Unconfined Compressive Strength, psf
			SILTY CLAY (CH)			
	1	P	Very soft, dark gray, with high organic material content in top 4 feet	73	-	-
		fv	Su(fv)=300psf			
5			(BAY MUD)			
	2	P	Su(uu)=175psf	34 35	77 78	270 -
			SILTY CLAY (CL-CH): Stiff, gray-brown	28	-	-
10						
	3	44*	SILTY CLAY (CL) Very stiff, brown and light gray mottled	24	102	5650
15	4	50*	Very stiff to hard	-	-	-
			BOTTOM OF BORING @ 16 feet			
20						
25						
30						

BOTTOM OF BORING @ 16 feet

- Salt pond water depth @ boring location = 2± feet
- 1± inch salt crust covering pond bottom

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

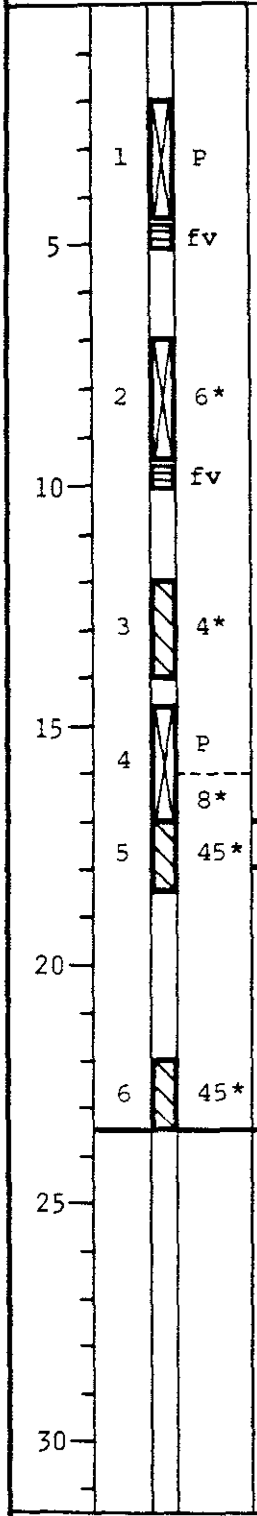
Log of Boring No. 7

Date Drilled: October 5 & 6, 1983
Type of Boring: 4-inch wash
Hammer Weight: 50 lbs.

Remarks: Exploratory boring advanced using barge-mounted portable drilling equipment

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 2½± feet



MATERIAL DESCRIPTION

SILTY CLAY (CH)

Very soft, dark gray, with high organic material content in top 3 feet

(BAY MUD)

SILTY CLAY (CL-CH): Stiff, gray-brown

SILTY CLAY (CL)

Very stiff, brown

Stiff, slightly sandy

BOTTOM OF BORING @ 23½ feet

- Salt pond water depth @ boring location = 3/4± ft.
- 1± inch salt crust covering pond bottom

Samples	Blows/Ft	Test	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
1	P	Su(uu)=160psf	55	67	-
	fv	Su(fv)=340psf			
2	6*	PI = 44 Su(uu)=230psf	65	60	-
	fv	Su(fv)=480psf			
3	4*		73	55	320
4	P 8*	Su(uu)=190psf	74	56	-
5	45*		20	109	4190
6	45*		21	107	2100

LABORATORY TESTS

Proj. No. 15945W

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Figure A-10

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 8

Date Drilled: September 24, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.
Remarks: Boring drilled on top of sludge lagoon levee
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 13½± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content, %	LABORATORY TESTS: Dry Density, pcf	LABORATORY TESTS: Unconfined Compressive Strength, psf
			SILTY CLAY FILL (CL-CH)			
	1 ▨	23	Moderately to well compacted, moist, gray-brown	24	99	7060
	2 ▨	14	} Very silty (CL-ML), brown	23	100	2970
5						
	3 ▨	27		23	100	4430
	4 ▨	18		19	108	4920
10						
			▽ 2 Days			
	5 ▨	22		20	101	3750
15			(FILL) ↑			
			▽ ATD SILTY CLAY (CL-CH) Very stiff, gray-brown mottled			
	6 ▨	18		26	100	4460
20			BOTTOM OF BORING @ 20 feet			
25						
30						

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Figure A-11

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 9

Date Drilled: September 26, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks:

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 8± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	LABORATORY TESTS Dry Density, pcf	LABORATORY TESTS Unconfined Compressive Strength, psf
	1	12	SOIL AND TRASH FILL Poorly compacted, damp, dark brown, with some wood, glass, etc.	-	-	-
			SILTY CLAY FILL (CL) Poorly compacted, moist, blue-gray			
5	2	5	▽ ATD TRASH FILL Wood, glass, rubber, etc., and some soil	-	-	-
10			(FILL) ↑			
15	3	31	SILTY CLAY (CL-CH) Stiff, gray-brown	29	95	3170
			SILTY CLAY (CL) Stiff, brown and gray mottled			
20	4	26		22	106	3020
			BOTTOM OF BORING @ 20 feet			
25						
30						

Proj. No. 15945W | **Woodward-Clyde Consultants** | Figure A-12

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 10

Date Drilled: September 24, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: Boring drilled on top of sludge lagoon levee

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 13½± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			SILTY CLAY FILL (CL-CH): Moderately compacted, damp, brown, with sand lenses			
	1	9	} Silty sand (SM) -#200 = 28%	-	-	-
	2	6	} Silty sand (SM)	-	-	-
5	3	27	SILTY CLAY FILL (CL-CH) Moderately to well compacted, moist, brown and gray	26	98	5990
	4	38		12	113	8140
10	5	32		22	104	7360
			↓ Less plastic (CL), dark and light brown mottled			
		▽ 2 Days				
			↑ (FILL)			
15	6	14	SILTY CLAY (CH) Stiff, brown-gray	27	93	2730
20	7	19	SILTY CLAY (CL-CH) Very stiff, brown and light gray mottled	-	-	-
			BOTTOM OF BORING @ 21½ feet (Dry at Time of Drilling)			
25						
30						

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Figure A-13

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 11

Date Drilled: September 26, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks:
(See Legend Sheet for sampler types and hammer weights)

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 2± feet			
	1	6	CLAYEY SILT (ML-CL) Soft, wet, brown PI = 6	31	90	930
5	2	6	SILTY CLAY (CH) Medium, moist to wet, brown-gray	32	89	1850
	3	6	↓ Stiff	27	93	2830
	4	17		24	101	4090
10			SILTY CLAY (CL-CH) Very stiff, moist, gray-brown			
			▽ ATD			
	5	16	↓ Less plastic (CL), stiff, brown and light gray mottled	25	101	2580
15						
	6	26	SILTY CLAY (CL): Very stiff to hard, brown	-	-	-
20			BOTTOM OF BORING @ 20 feet			
25						
30						

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Figure A-14

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 12

Date Drilled: September 24, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: Boring drilled on top of sludge lagoon levee
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 13½± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1 ▨	13	SILTY CLAY FILL (CL) Moderately to well compacted, moist, dark brown, interbedded with thin layers of Clayey Silt (ML)	17	110	4040
5	2 ▨	26	PI = 15	17	111	2530
	3 ▨	37		17	113	7750
10	4 ▨	26	(FILL)	15	118	9350
			SILTY CLAY (CL-ML) ▽ 2 Days Medium, wet, gray-brown and rust mottled, very silty			
15	5 ▨	7		38	82	1120
			▽ ATD More plastic (CL-CH), stiff to very stiff			
20	6 ▨	24		-	-	-
			BOTTOM OF BORING @ 20 feet			
25						
30						

Proj. No. 15945W

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Figure A-15

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 13

Date Drilled: September 26, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 8± feet

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	11	SILTY CLAY FILL (CL-ML) Moderately compacted, moist, dark gray and brown with high organic material content, including fragments of nut shells (FILL)	31	77	4090
	2	12	SANDY SILT (ML) Loose to medium dense, wet, gray, with interbedded clay seams -#200 = 55%	24	101	1020
5						
	3	5	Grading to Silt (ML), loose, brown-gray, slightly clayey	-	-	-
	4	2	▽ ATD	-	-	-
10						
	5	4	SILTY CLAY (CH) Very soft, wet, dark gray, with organic material	53	66	340
15			(BAY MUD)			
	6	4		28	94	290
20			SANDY SILT (ML) Loose, brown, slightly clayey			
	7	6		-	-	-
			BOTTOM OF BORING @ 22 feet			
25						
30						

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 14

Date Drilled: September 26, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks:

(See Legend Sheet for sampler types and hammer weights)

				LABORATORY TESTS		
Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 4± feet			
			SILTY CLAY (CL)			
	1 ▨	7	Medium, moist to wet, brown and rust mottled, with lenses of Clayey Silt (ML) and Silty Sand (SM) PI = 20	33	87	1070
	2 ▨	7	▽ ATD	30	92	970
5			SILTY CLAY (CL-CH) Medium, dark brown-gray mottled			
	3 ▨	4		27	97	1170
	4 ▨	4	SILTY CLAY (CH)	31	92	800
10			Soft, dark gray			
			(BAY MUD)			
	5 ▨	3	↓ With lenses of Silty Sand (SM)	41	77	590
15						
	6 ▨	28	SILTY CLAY (CL) Very stiff, brown	-	-	-
20			BOTTOM OF BORING @ 20 feet			
25						
30						

Proj. No. 15945W

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Figure A-17

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 15

Date Drilled: September 26, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 8± feet

Depth, Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	9	CLAYEY SILT - SILTY CLAY FILL (ML-CL) Poorly compacted, moist, brown with some sandy zones	17	97	1220
	2	10	SILTY CLAY (CH) Stiff, moist, dark gray-brown	24	100	3260
5	3	6	Less plastic (CL), medium, brown	-	-	-
	4	10	▽ ATD	28	94	1850
10						
	5	20	SILTY CLAY (CL) Very stiff, brown	18	111	5900
15						
	6	18	Becoming sandy, light brown	-	-	-
20	7	14	SAND (SP-SM) Medium dense, brown	-	-	-
			SILTY CLAY (CL) Stiff, brown and light gray mottled			
			BOTTOM OF BORING @ 21½ feet			
25						
30						

Proj. No. 15945W

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Figure A-18

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 16

Date Drilled: September 20, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks:

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 10± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION		Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			SILTY CLAY (CL) Stiff, moist, brown				
	1	10		PI = 14	21	101	-
5	2	6	With seams of Clayey Silt (ML), soft, wet, gray-brown mottled	PI = 13	37	84	-
	3	5	SILTY SAND (SM) Loose, wet, gray-brown	-#200 = 45%	-	-	-
			SILTY CLAY (CH) Medium, moist, dark brown-gray				
	4	13	Stiff		26	94	2970
10			▽ ATD				
15	5	21	SILTY CLAY (CL) Very stiff, brown	PI = 20	20	108	4730
20	6	17			23	104	3850
	7	26			23	105	6280
25			BOTTOM OF BORING @ 25 feet				
30							

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Figure A-19

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 17

Date Drilled: September 21, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.
Remarks: Boring drilled on top of sludge lagoon levee
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 16± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION		LABORATORY TESTS	
				Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			SILTY CLAY - CLAYEY SILT FILL (CL-ML) Well compacted, damp, gray-brown with thin layers of Silty Sand (SM) and scattered fine to medium gravel			
	1	29	PI = 8	8	122	8920
	2	32	Moist PI = 10	15	114	9500
5						
	3	27		19	106	8090
	4	15	(FILL)	18	112	10,100
10			ATD SANDY CLAYEY SILT (ML) Stiff, brown			
			SILTY SAND (SM)			
	5	12	-#200 = 40%	-	-	-
15			Medium dense, dark brown			
	6	21		-	-	-
20						
	7	12		-	-	-
			SILTY CLAY (CL-CH) Very stiff to hard, brown and light gray mottled			
25						
	8	35		-	-	-
			BOTTOM OF BORING @ 26½ feet			
30						

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Figure A-20

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 18

Date Drilled: September 20, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 10± feet

Depth, Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	 Dry Density, pcf	 Unconfined Compressive Strength, psf
			SILTY CLAY (CL)			
	1 ▨	6	Medium, moist, brown, very silty PI = 15	27	91	1660
	2 ▨	5	CLAYEY SILT (ML): Soft, wet, brown, slightly sandy ▽ ATD -#200 = 62%	29	91	-
5			SILTY SAND (SM) Loose, brown			
	3 ▯	8		-	-	-
			SILTY CLAY (CH) Stiff, gray-brown			
	4 ▨	14		27	96	2870
10			Slightly less plastic (CL-CH), very stiff ↓			
	5 ▨	16		23	102	4380
			SILTY CLAY (CL) Very stiff, brown and light gray mottled			
15						
	6 ▨	18		20	109	4670
			BOTTOM OF BORING @ 16½ feet			
20						
25						
30						

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Figure A-21

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 19

Date Drilled: September 20, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.
Remarks: Boring drilled on top of sludge lagoon levee
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 16± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	17	SILTY CLAY - CLAYEY SILT FILL (CL-ML) Well compacted, moist, brown	12	120	13,790
5	2	21	PI = 9	13	109	4820
	3	14	CLAYEY SILT FILL (ML-CL): Moderately compacted, moist to wet, brown, slightly sandy (FILL)	19	105	2100
10	4	10	SILTY CLAY (CL) Medium, moist to wet, gray-brown, with roots ▽ 9-22-83	25	98	1070
15	5	9	▽ ATD SILTY CLAY (CH) Medium, dark gray-brown	29	93	1710
20	6	16	SILTY CLAY (CL) Very stiff, brown and gray mottled with tan streaks Free groundwater encountered during drilling	22	105	4530
25	7	10	SILTY CLAY (CL) Stiff, brown and light gray mottled, very silty with some fine sand	-	-	-
30	8	17		-	-	-

BOTTOM OF BORING @ 30 feet

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Figure A-22

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 20

Date Drilled: September 20, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

				LABORATORY TESTS		
Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 11± feet			
			SANDY SILT (ML) Very loose, moist, brown			
	1	6	PI : Non-plastic	22	100	2140
			CLAYEY SILT - SILTY CLAY (ML-CL) Stiff, moist, brown Soft, wet			
5	2	4	PI = 10	31	91	830
			ATD SILTY SAND (SM): Loose, brown			
	3	12		-	-	-
			SILTY CLAY (CH) Medium, dark gray-brown			
10	4	12	PI = 43	28	92	1610
			SILTY CLAY (CL-CH) Very stiff, gray-brown mottled with some sand inclusions			
15	5	20		21	106	5850
20	6	16		-	-	-
	7	-		-	-	-
			BOTTOM OF BORING @ 23 feet			
25						
30						

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Figure A-23

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 21

Date Drilled: September 20, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: Boring drilled on top of sludge lagoon levee

(See Legend Sheet for sampler types and hammer weights)

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS		
				Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 15½± feet			
	1	17	SILTY CLAY - CLAYEY SILT FILL (CL-ML) Well compacted, moist, gray-brown	13	113	6670
5	2	31	With partings of sandy clayey Silt (ML) PI = 11	15	113	5900
	3	33	(FILL)	16	112	7890
10	4	8	SILTY CLAY (CL-ML) Medium, moist to wet, gray-brown with roots	22	98	1220
15	5	11	SILTY CLAY (CH) Stiff, moist, dark gray-brown	25	97	3410
20	6	20	SILTY CLAY (CL-CH) Very stiff, moist, brown and light gray mottled	-	-	-
			▽ ATD			
25	7	22		-	-	-
30	8	12	SILTY SAND (SM): Medium dense, brown	-	-	-
			BOTTOM OF BORING @ 30 feet			

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Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 22

Date Drilled: September 19, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks:

(See Legend Sheet for sampler types and hammer weights)

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 10½± feet			
			SILTY CLAY - CLAYEY SILT (CL-ML) Medium, moist, brown, with lenses of Sandy Silt (ML)			
	1 ▨	7	PI = 8	24	96	1270
	2 ▥	4	▽ ATD GRAVELLY SAND (SW) Very loose, wet, gray-brown + #4 = 31% -#200 = 4%	-	-	-
5						
	3 ▥	9	Increasing fines content (SM), loose + #4 = 34% -#200 = 21%	-	-	-
	4 ▨	15		30	91	1610
10			SILTY CLAY (CH) Medium, dark gray-brown to brown-gray			
	5 ▨	14	Stiff	27	96	2240
15						
			SILTY CLAY (CL) Very stiff, brown and gray mottled			
	6 ▨	20		-	-	-
20			BOTTOM OF BORING @ 20 feet			
25						
30						

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Figure A-25

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 23

Date Drilled: September 13 & 19, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 14½± feet

Depth, Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content, %	LABORATORY TESTS: Dry Density, pcf	LABORATORY TESTS: Unconfined Compressive Strength, psf
			SILTY SAND FILL (SM) Poorly compacted, moist, brown -#200 = 39%			
	1	4	(FILL)	15	92	590
			SILTY CLAY - CLAYEY SILT (CL-ML) Soft, moist, brown			
5	2	4	Medium	21	101	1510
	3	5	Grading to sandy clayey Silt (ML-CL) ▽ ATD	24	95	-
	4	7	SANDY SILT - SILTY SAND (ML-SM) Loose, brown -#200 = 53%	-	-	-
10	5	11	-#200 = 61%	-	-	-
	6	6	SILTY CLAY (CH) Medium, dark gray mottled	-	-	-
15			SILTY CLAY (CL) Medium, gray and olive mottled Becoming more plastic (CL-CH), with traces of organic material			
	7	10		27	98	1170
20			SILTY CLAY (CH) Soft, dark gray, with traces of organic material			
	8	-	(BAY MUD)	52	69	490
25						
	9	-	SILTY SAND (SM) Loose to medium dense	-	-	-
30			BOTTOM OF BORING @ 30 feet			

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Figure A-26

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 24

Date Drilled: September 20, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks:

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 12± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	LABORATORY TESTS Dry Density, pcf	LABORATORY TESTS Unconfined Compressive Strength, psf
			SILTY CLAY - CLAYEY SILT (CL-ML) Medium, moist, brown, very silty			
	1 ▨	5	PI = 9	22	98	1320
			SILTY SAND (SM) Very loose, moist, brown			
	2 ▨	2	-#200 = 25%	-	-	-
5			Grading finer			
	3 ▯	5	SILTY CLAY (CL) Soft to medium, wet, dark gray-brown ▽ ATD	-	-	-
	4 ▨	12	SILTY CLAY (CH) Stiff, dark brown-gray	24	97	3260
10						
	5 ▨	18		23	101	3170
	6 ▨	12	SILTY CLAY (CL) Stiff, light brown mottled	26	97	2440
15						
	7 ▨	10		-	-	-
20			BOTTOM OF BORING @ 20 feet			
25						
30						

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 25

Date Drilled: September 19, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: Boring drilled on top of interim flood control levee

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 14½± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			CLAYEY SILT FILL (ML-CL) Moderately compacted, damp, brown, with fragments of decomposing wood (FILL)			
	1	12	**SANDY CLAYEY SILT** (ML) Medium, moist, brown	13	105	1460
5	2	6		17	100	1460
	3	3	**SILTY CLAY - CLAYEY SILT** (CL-ML) Soft, wet, dark brown PI = 10	29	89	680
10	4	7	▽ ATD **SANDY SILT** (ML) Loose, wet, brown, with interbeds of Silty Sand (SM) -#200 = 45%	-	-	-
	5	13		-	-	-
15	6	17	**SILTY SAND** (SM) Medium dense, dark brown + #4 = 8% -#200 = 19%	-	-	-
	7	27	**SANDY GRAVEL** (GP-GM) Medium dense, dark gray + #4 = 49% -#200 = 8%	-	-	-
20	8	20		-	-	-
			BOTTOM OF BORING @ 21½ feet			
25						
30						

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Figure A-28

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 26

Date Drilled: September 19, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: Boring drilled on top of interim flood control levee

(See Legend Sheet for sampler types and hammer weights)

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content, %	LABORATORY TESTS: Dry Density, pcf	LABORATORY TESTS: Unconfined Compressive Strength, psf
			Surface Elevation: 16± feet			
			S A N D Y S I L T F I L L (ML)			
	1	12	Poorly compacted, damp, brown, with slight clay content	8	87	540
			(F I L L)			
5	2	11		9	87	2530
			C L A Y E Y S I L T (ML-CL) Stiff, damp, brown			
	3	6	Moist	20	96	3260
			Soft to medium, wet, brown and gray mottled			
	4	12		30 22	93 100	970 -
10	5	6	▽ ATD SILTY SAND (SM): Loose, brown Grades to Sand (SP-SM) -#200 = 10%	-	-	-
			S A N D Y C L A Y E Y S I L T (ML) Medium, gray			
15	6	3	Grading to Sandy Clay (CL-SC) with silt lenses PI = 13	25	100	1320
20	7	10		-	-	-
	8	19	S A N D (SP) Medium dense, dark gray with scattered fine gravel	-	-	-
25			Increasing gravel content			
30			BOTTOM OF BORING @ 28½ feet			

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Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 27

Date Drilled: September 19, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks:

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 17± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION		LABORATORY TESTS Moisture Content, %	LABORATORY TESTS Dry Density, pcf	LABORATORY TESTS Unconfined Compressive Strength, psf
			SILTY CLAY - CLAYEY SILT (CL-ML) Medium, damp to moist, brown with some sandy zones				
	1	7		PI = 8	15	95	1320
			Moist				
5	2	6		PI = 10 -#200 = 79%	21	97	-
	3	5	Very silty (ML-CL)	PI = 6	22	99	1710
			SILTY CLAY (CH) Very stiff, moist, dark gray-brown				
	4	14			23	102	4340
10							
			Slightly less plastic (CL-CH)				
			▽ ATD				
	5	19			-	-	-
15			SILTY CLAY (CL) Very stiff, brown mottled				
	6	7			-	-	-
20			SANDY SILT - SILTY SAND (ML-SM) Loose, brown	-#200 = 49%			
	7	10			-	-	-
25			Grading coarser, Silty Sand (SM-SP)				
			BOTTOM OF BORING @ 25½ feet				
30							

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Figure A-30

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 28

Date Drilled: September 16, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.
Remarks: Boring drilled on top of interim flood control levee

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 25± feet

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS		
				Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			SANDY CLAY FILL (CL-SC)			
	1	11	Moderately compacted, damp, brown, with rock fragments	-	-	-
	2	11	PI = 11 -#200 = 53%	14	113	1560
5	3	17	(FILL)	15	116	2530
			CLAYEY SAND (SC-SM)			
	4	24	Medium dense, moist, dark brown-gray, with scattered fine gravel + #4 = 4% -#200 = 41%	13	120	4000
			Sandy Clay (CL)			
10	5	11	Grades to Silty Sand (SM) with scattered gravel, loose + #4 = 7% -#200 = 35%	12	113	-
			SANDY CLAY (CL-ML)			
15	6	9	Medium, wet, dark brown-gray Small piece of wood	19	108	-
			▽ ATD			
20	7	17	Silty Sand (SM), medium dense with gravelly lenses + #4 = 27% -#200 = 14%	-	-	-
			(CREEK BED DEPOSITS)			
			SILTY CLAY (CL) Medium, light brown			
25	8	22		-	-	-
			SILTY SAND (SM) Medium dense, brown			
	9	27		-	-	-
			GRAVELLY SAND (SW) Medium dense, dark gray and brown			
30	10	34	BOTTOM OF BORING @ 31½ feet	-	-	-

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Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 29

Date Drilled: September 16, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks:

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 22± feet

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	11	SANDY SILT (ML) Medium dense, dry to damp, light brown with some clay	7	97	1610
	2	9	Loose, increasing sand content	6	90	1020
5						
	3	18	CLAYEY SILT - SILTY CLAY (ML-CL) Stiff, damp, light brown, pitted	9	99	-
	4	16	SILTY SAND (SM)	-	-	-
10	5	14	Medium dense, damp, brown -#200 = 25%	-	-	-
	6	30	With some fine gravel, dark brown + #4 = 5% -#200 = 25%	-	-	-
15						
			▽ ATD			
	7	24	SANDY GRAVEL (GW) Medium dense, dark brown	-	-	-
20						
	8	-	SANDY CLAY (CL): Stiff, brown	-	-	-
25			BOTTOM OF BORING @ 25 feet			
30						

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 30

Date Drilled: September 16, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.
Remarks:
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 20± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	8	SILTY CLAY - CLAYEY SILT (CL-ML) Stiff, damp to moist, brown, with some fine sand PI = 10 -#200 = 84%	13	96	2100
	2	8	} Silty Sand (SM) lens	18	93	1510
5	3	9	SILTY CLAY (CL) Medium, moist, brown mottled PI = 14 Dark brown, with some gravel	22	100	-
	4	18	SILTY CLAY (CH) Very stiff, moist, dark gray-brown	23	102	4290
10	5	17		-	-	-
15	6	12	▽ ATD SILTY CLAY (CL) Medium, light brown, with some sand	22	104	1460
20	7	24	SAND (SP) Medium dense, dark brown With gravel	-	-	-
			BOTTOM OF BORING @ 21½ feet			
25						
30						

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Figure A-33

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. 31

Date Drilled: September 16, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks:

(See Legend Sheet for sampler types and hammer weights)

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	 Dry Density, pcf	 Unconfined Compressive Strength, psf
			Surface Elevation: 18½± feet			
			CLAYEY SILT (ML-CL)			
	1	9	Stiff, damp, light brown, with roots	10	87	3310
	2	9	Grading to Sandy Silt (ML) loose, brown and rust, with trace clay -#200 = 78%	12	92	-
5	3	13	SILTY CLAY (CL-ML) Medium, moist, dark brown-gray and rust mottled, with interbeds of less plastic silt and fine sand	-	-	-
	4	11	Silty Sand (SM) lens	17	114	1120
10	5	13	Stiff	24	100	3020
			SILTY CLAY (CL-CH)			
15	6	15	▽ ATD Very stiff, moist, light brown and gray mottled	23	102	4090
20	7	11	Less plastic (CL), stiff	-	-	-
			BOTTOM OF BORING @ 21½ feet			
25						
30						

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Figure A-34

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. P-1

Date Drilled: July 2, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: 2-inch diameter slotted PVC standpipe piezometer installed to 20 feet

(See Legend Sheet for sampler types and hammer weights)

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content, %	LABORATORY TESTS: Dry Density, pcf	LABORATORY TESTS: Unconfined Compressive Strength, psf
			Surface Elevation: 8.6± feet (Top of Piezometer)			
			CLAYEY SILT (ML) Medium, dry, gray-brown			
			SILTY CLAY (CL) Medium, moist, brown			
	1	8	Interbedded with lenses and layers of Silty Sand (SM), Sandy Silt (ML), and Clayey Silt (ML)	28	94	-
5						
			▽ ATD			
	2	5	CLAYEY SAND (SC): Loose, wet, dark gray	67	58	540
10			SILTY CLAY (CH) Soft, dark gray, with some organic material			
			(BAY MUD)			
	3	3		51	72	290
15			Becoming sandy, very soft			
			SAND (SW-SM) Medium dense, dark gray			
	4	24		-	-	-
20			Becoming gravelly +#4 = 28% -#200 = 10%			
	5	8		-	-	-
25			BOTTOM OF BORING @ 25 feet			
30						

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Figure A-35

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. P-2

Date Drilled: July 2, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: 2-inch diameter slotted PVC standpipe piezometer installed to 20 feet
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 9.6± feet (Top of Piezometer)

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			SANDY SILT (ML) Loose, dry to damp, brown Moist to wet, more silty			
	1	5	SANDY SILT - SILTY SAND (ML-SM) Loose, wet, brown	-	-	-
5						
			▽ 7-5-83			
	2	15	SILTY CLAY (CH) Very stiff, moist, dark brown-gray	27	96	4000
10						
			Slightly less plastic (CL-CH), brown, with sand inclusions			
	3	19		24	104	4970
15						
	4	16	With gradational zones of very silty clay (CL)	21	105	4730
20						
			With light brown cemented nodules			
	5	27		21	107	6190
25			BOTTOM OF BORING @ 25 feet			
30						

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. P-3

Date Drilled: July 2, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: 2-inch diameter slotted PVC standpipe piezometer installed to 20 feet

(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 11.3± feet (Top of Piezometer)

Depth, Ft	Samples	Blows/Ft	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content, %	LABORATORY TESTS: Dry Density, pcf	LABORATORY TESTS: Unconfined Compressive Strength, psf
	1	8	SILTY CLAY (CL) Medium, damp, brown PI = 14 Moist	17	101	-
5	2	10	Wet ▽ATD ▽7-5-83 SILTY CLAY (CH) Stiff, dark brown-gray	28	94	2920
10	3	23	Slightly less plastic (CL-CH), very stiff, gray-brown, with quartz sand inclusions	23	104	5700
15	4	22	SILTY CLAY (CL) Stiff to very stiff, brown mottled	22	104	3850
20	5	17		20	107	4190
25	6	-	SILTY CLAY (CL-CH) Hard, gray-brown with some light brown cemented nodules	19	109	8180
30			BOTTOM OF BORING @ 27 feet			

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Figure A-37

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. P-4

Date Drilled: July 2, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: 2-inch diameter slotted PVC standpipe piezometer installed to 20 feet
(See Legend Sheet for sampler types and hammer weights)

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS		
				Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 11.4± feet (Top of Piezometer)			
			CLAYEY SILT (ML-CL) Soft, damp, brown			
	1	6	Less plastic (ML) with some fine sand, wet	26	95	630
5						
			▽ 7-5-83			
			SILTY CLAY (CL-CH) Stiff, moist, dark brown			
	2	18	Very stiff, with trace of sand	19	108	5120
			▽ ATD			
10						
	3	13	SILTY CLAY (CL) Stiff, brown mottled	24	101	2440
15						
	4	20	Very stiff	20	110	5360
20						
			Increasing silt content, stiff			
	5	11	SILTY CLAY (CL-CH): Stiff, light brown	25	100	3170
25			BOTTOM OF BORING @ 24½ feet			
30						

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Figure A-38

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. P-5

Date Drilled: July 2, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: 2-inch diameter slotted PVC standpipe piezometer installed to 20 feet

(See Legend Sheet for sampler types and hammer weights)

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS		
				Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 13.8± feet (Top of Piezometer)			
			CLAYEY SILT (ML) Medium, moist, brown with fine sand			
	1	8	Slightly more clayey, stiff	18	107	2436
5			▽ ATD SILTY CLAY (CL) Medium, moist, gray-brown			
	2	9	SILTY CLAY (CH) Medium, moist to wet, dark brown-gray	30	92	1120
			▽ 7-5-83 Stiff			
10	3	25	Slightly less plastic (CL-CH), very stiff, brown-gray	25	100	5700
			Light brown-gray			
15	4	32	SANDY CLAY (CL) Very stiff to hard, light brown and gray mottled	17	113	6580
20	5	26	SILTY SAND (SM-SC) Medium dense, brown and gray, with some clay	-	-	-
			SAND (SP-SM) Medium dense, rust brown with some gray			
25	6	8	SANDY SILT (ML) Loose, gray	-	-	-
			BOTTOM OF BORING @ 27 feet			
30						

Proj. No. 15945W

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Figure A-39

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Boring No. P-6

Date Drilled: July 2, 1983
Type of Boring: 6-inch auger
Hammer Weight: 140 lbs.

Remarks: 2-inch diameter slotted PVC standpipe piezometer installed to 20 feet
(See Legend Sheet for sampler types and hammer weights)

Surface Elevation: 19.3± feet (Top of Piezometer)

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content, %	LABORATORY TESTS: Dry Density, pcf	LABORATORY TESTS: Unconfined Compressive Strength, psf
			SILTY CLAY (CL-ML) Stiff, damp, brown			
	1	9	SANDY SILT (ML) Loose, moist, brown -#200=73%	23	91	-
			▽ ATD			
5			With zones of Clayey Silt (ML-CL)			
	2	10	SANDY SILT (ML): Loose, wet, dark gray-brown	-	-	-
10			Grading to Silty Sand (SM) -#200 = 48% ▽ 7-5-83			
			Becoming clayey, with gravel			
	3	12		24	99	3220
15			SILTY CLAY (CL) Stiff, brown and gray mottled			
	4	26	Grading to sandy Clayey Silt (ML)	-	-	-
20			SILTY SAND (SM) Medium dense, dark brown			
	5	16	-#200 = 33%	-	-	-
25			BOTTOM OF BORING @ 25 feet			
30						

Proj. No. 15945W

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Figure A-40

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Exploratory Pit No. 1

Date Excavated: December 5, 1983

Type of Excavation: Backhoe - 18" Bucket

Remarks:

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS		
				Moisture Content,%	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 8± feet			
			SILTY CLAY (CL)			
	1	--	Medium to stiff, moist, brown, very silty, with sandy lenses PI = 8	21	-	-
	2	--		23	-	-
5	3	--	Wet	31	-	-
			ATE * SANDY SILT (ML) Loose, saturated			
10			BOTTOM OF PIT @ 8 feet			
15						
20						
25						
30						

* Seepage at Time of Excavation (ATE)

Proj. No. 15945W

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Figure A-41

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Exploratory Pit No. 2

Date Excavated: December 5, 1983

Type of Excavation: Backhoe - 18" Bucket

Remarks:

Surface Elevation: 10± feet

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	--	SILTY CLAY (CL) Medium, moist, brown, very silty PI = 10	22	-	-
	2	--	Wet	24	-	-
			SILTY SAND (SM): Loose, brown, saturated			
	3	--	∇* ATE SILTY CLAY (CL) Medium, wet, brown	19	-	-
5			BOTTOM OF PIT @ 5 feet			
10						
15						
20						
25						
30						

* Seepage at Time of Excavation (ATE)

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Exploratory Pit No. 3

Date Excavated: December 5, 1983
Type of Excavation: Backhoe - 18" Bucket
Remarks:

Surface Elevation: 11± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1	--	SILTY CLAY (CL): Medium to stiff, moist, gray-brown, very silty PI = 9	18	-	-
	2	--	CLAYEY SILT - SILTY CLAY (ML-CL) Medium, wet, brown, slightly sandy	20	-	-
	3	--		32	-	-
			▽ * SILTY SAND (SM) Loose, saturated, dark brown ATE			
5						

BOTTOM OF PIT @ 5 feet

* Seepage at Time of Excavation (ATE)

10

15

20

25

30

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Exploratory Pit No. 4

Date Excavated: December 5, 1983
Type of Excavation: Backhoe - 18" Bucket
Remarks:

Depth, Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 12½± feet			
	1 ▨	--	SILTY CLAY (CL) Stiff, moist, dark brown PI = 12	14	-	-
	2 ▨	--	SILTY CLAY - CLAYEY SILT (CL-ML): Medium, moist, brown	22	-	-
	3 ▨	--	}Silty Sand (SM-ML) ⊤ Wet	22	-	-
			▽ * ⊤ Soft, saturated			
5			ATE SILTY SAND (SM) Loose, dark brown			
10			BOTTOM OF PIT @ 6 feet			
15						
20						
25						
30						

* Seepage at Time of Excavation (ATE)

Proj. No. 15945W

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Figure A-44

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Exploratory Pit No. 5

Date Excavated: December 5, 1983
Type of Excavation: Backhoe - 18" Bucket
Remarks:

Surface Elevation: 18± feet

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content, %	LABORATORY TESTS: Dry Density, pcf	LABORATORY TESTS: Unconfined Compressive Strength, psf
	1 ▨	--	SILTY CLAY (CL) PI = 11 Medium to stiff, moist, brown, with some very silty zones	19	-	-
	2 ▨	--		18	-	-
	3 ▨	--		19	-	-
5	4 ▨	--	CLAYEY SILT (ML): Medium, moist to wet, brown Grades to Sandy Silt (ML), loose, caved during excavation	19	-	-
			BOTTOM OF PIT @ 6½ feet (Dry at Time of Excavation)			
10						
15						
20						
25						
30						

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Exploratory Pit No. 6

Date Excavated: December 5, 1983
Type of Excavation: Backhoe - 18" Bucket
Remarks:

Surface Elevation: 14± feet

Depth, Ft.	Samples	Blows/Ft	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	LABORATORY TESTS Dry Density, pcf	LABORATORY TESTS Unconfined Compressive Strength, psf
	1 ▨	--	SILTY CLAY (CL): Medium to stiff, moist, gray-brown, very silty PI = 11	18	-	-
	2 ▨	--	SILTY CLAY (CL) Stiff to very stiff, damp, light brown, very silty	8	-	-
	3 ▨	--	PI = 9	13	-	-
5	4 ▨	--	SANDY SILT (ML): Medium dense, moist, brown ↓ Grades to Silty Sand (SM)	16	-	-
	5 ▨	--	SILTY CLAY (CL) Medium, wet, dark brown, very silty	22	-	-
			BOTTOM OF PIT @ 6½ feet (Dry at Time of Excavation)			
10						
15						
20						
25						
30						

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Exploratory Pit No. 7

Date Excavated: December 5, 1983
Type of Excavation: Backhoe - 18" Bucket
Remarks:

Surface Elevation: 11½± feet

Depth, Ft	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
	1 ☒	--	SANDY CLAY (CL): Medium to stiff, moist, dark brown, with scattered gravel PI = 11	16	-	-
	2 ☒	--	SILTY CLAY - CLAYEY SILT (CL-ML) Medium, wet, brown	26	-	-
	3 ☒	--	▽ * ATE ⊤ Saturated, sandy	31	-	-
			SILTY SAND (SM): Loose, dark brown			
5			BOTTOM OF PIT @ 4 feet			
10			* Seepage at Time of Excavation (ATE)			
15						
20						
25						
30						

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Figure A-47

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Exploratory Pit No. 8

Date Excavated: December 5, 1983
Type of Excavation: Backhoe - 18" Bucket
Remarks:

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation: 10± feet			
	1 ▨	--	SILTY CLAY FILL (CL-CH) Poorly compacted, moist to wet, brown, with traces of wood and roots PI = 25	34	-	-
	2 ▨	--	(FILL)	33	-	-
			SANDY SILT (ML): Loose, wet, gray-brown			
			▽ * ATE SILTY CLAY (CL): Medium, saturated, light brown mottled			
5			BOTTOM OF PIT @ 5 feet			
10						
15						
20						
25						
30						

* Seepage at Time of Excavation (ATE)

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Figure A-48

Project: COYOTE CREEK IMPROVEMENTS
Santa Clara County, California

Log of Exploratory Pit No. 9

Date Excavated: December 5, 1983

Type of Excavation: Backhoe - 18" Bucket

Remarks:

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	LABORATORY TESTS: Moisture Content, %	LABORATORY TESTS: Dry Density, pcf	LABORATORY TESTS: Unconfined Compressive Strength, psf
			Surface Elevation: 7± feet			
			SILTY CLAY (CL) AND TRASH FILL Uncompacted, consisting of rug, metal, organic material, etc. in a clayey soil matrix ▽* ATE (FILL)			
5			CLAYEY SILT (ML): Soft, light brown and rust, with some fine sand			
			SILTY SAND (SM): Loose, brown			
			SILTY CLAY (CH) Medium, gray and rust mottled			
10			BOTTOM OF PIT @ 8½ feet			
			* Seepage at Time of Excavation (ATE)			
15						
20						
25						
30						

Proj. No. 15945W

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Figure A-49

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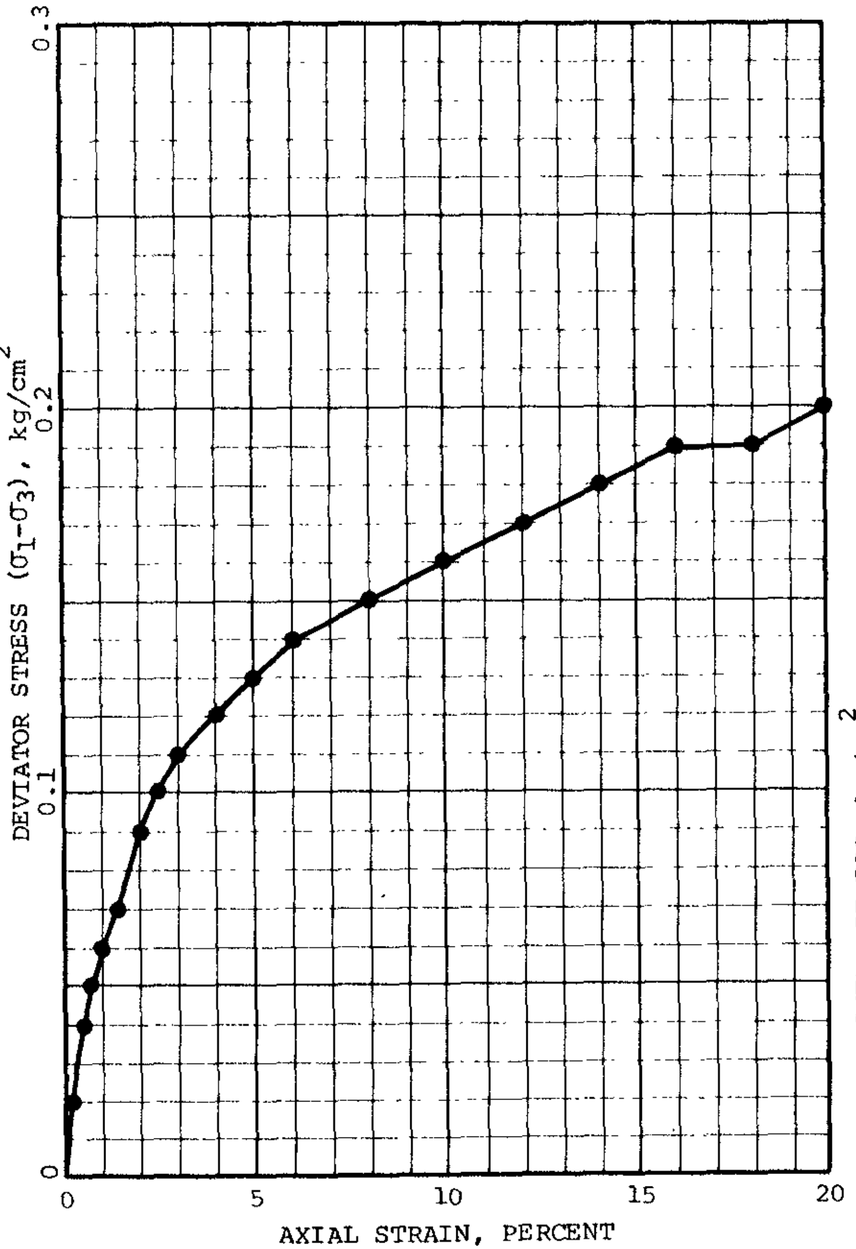
Project No. 15945W

TRIAXIAL COMPRESSION TEST

COYOTE CREEK IMPROVEMENTS

Figure A-50

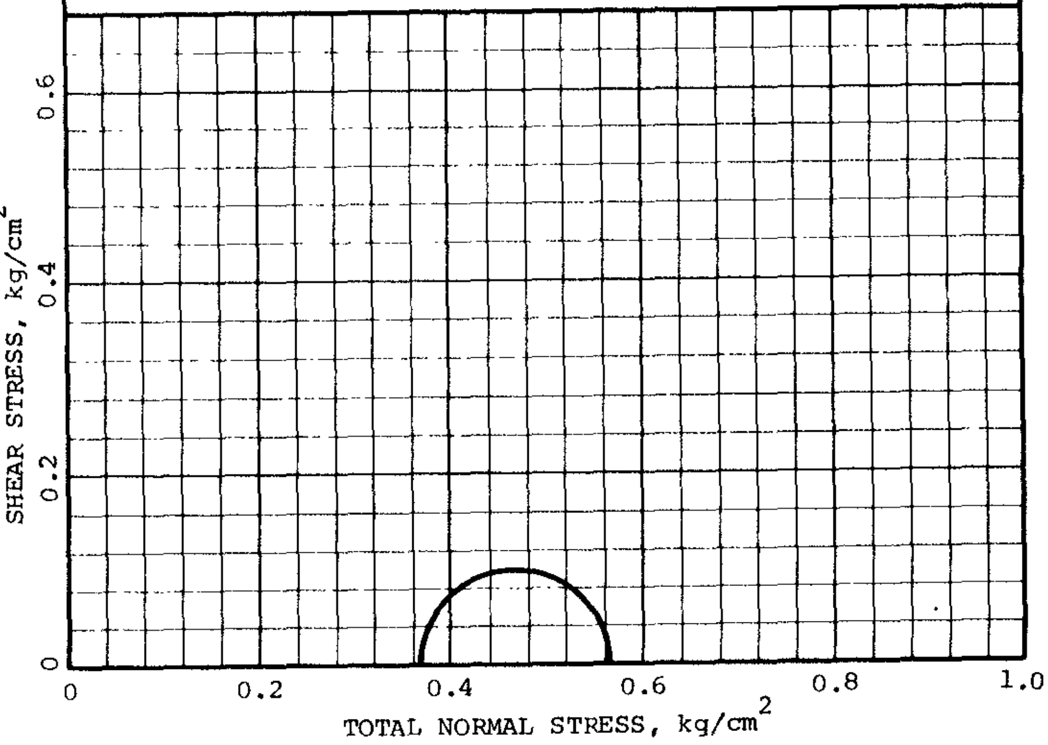


KEY

DEVIATOR STRESS, $\sigma_1 - \sigma_3$

SPECIMEN: Sample 6-2 (7½'-8')
TYPE OF SPECIMEN: 3-IN. DIAMETER SHELBY TUBE
TYPE OF TEST: UNCONSOLIDATED-UNDRAINED

SPECIMEN AND SYMBOL	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)	CONFINING PRESSURE (kg/cm^2)	DEVIATOR STRESS PEAK (kg/cm^2)	PORE PRESSURE AT FAILURE
6-2 ●	35.18	78.10	0.366	0.20	—



KEY

TOTAL STRESS

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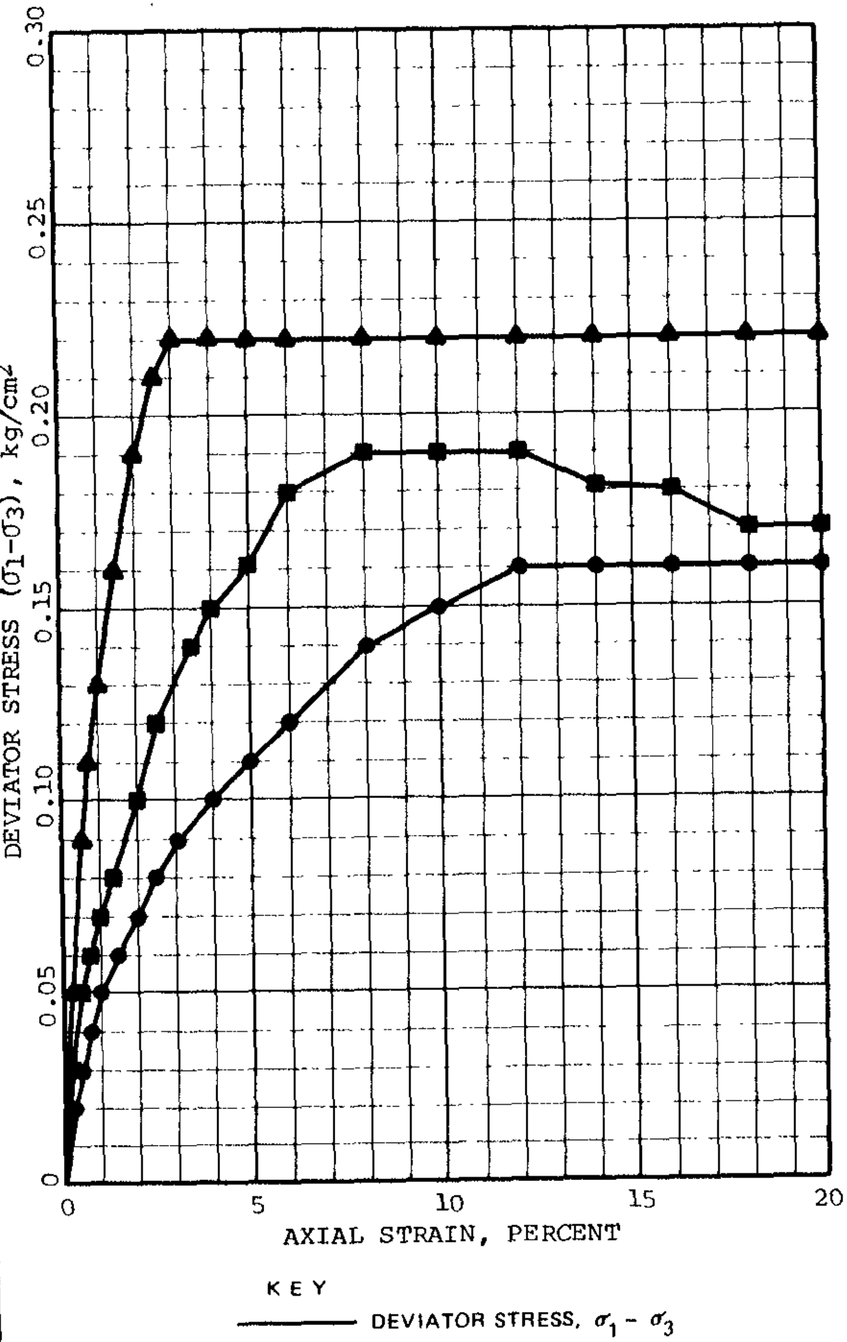
TRIAXIAL COMPRESSION TEST

COYOTE CREEK IMPROVEMENTS

Figure A-51

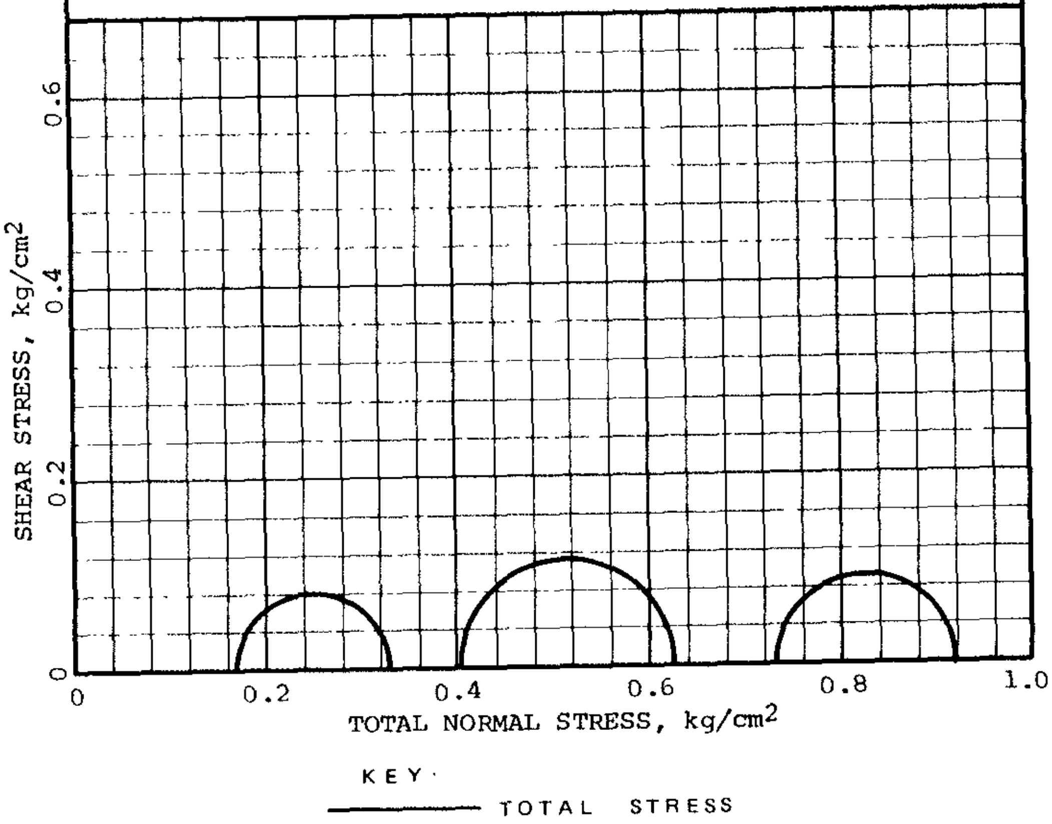


SPECIMEN: Samples 7-1, 7-2, and 7-3

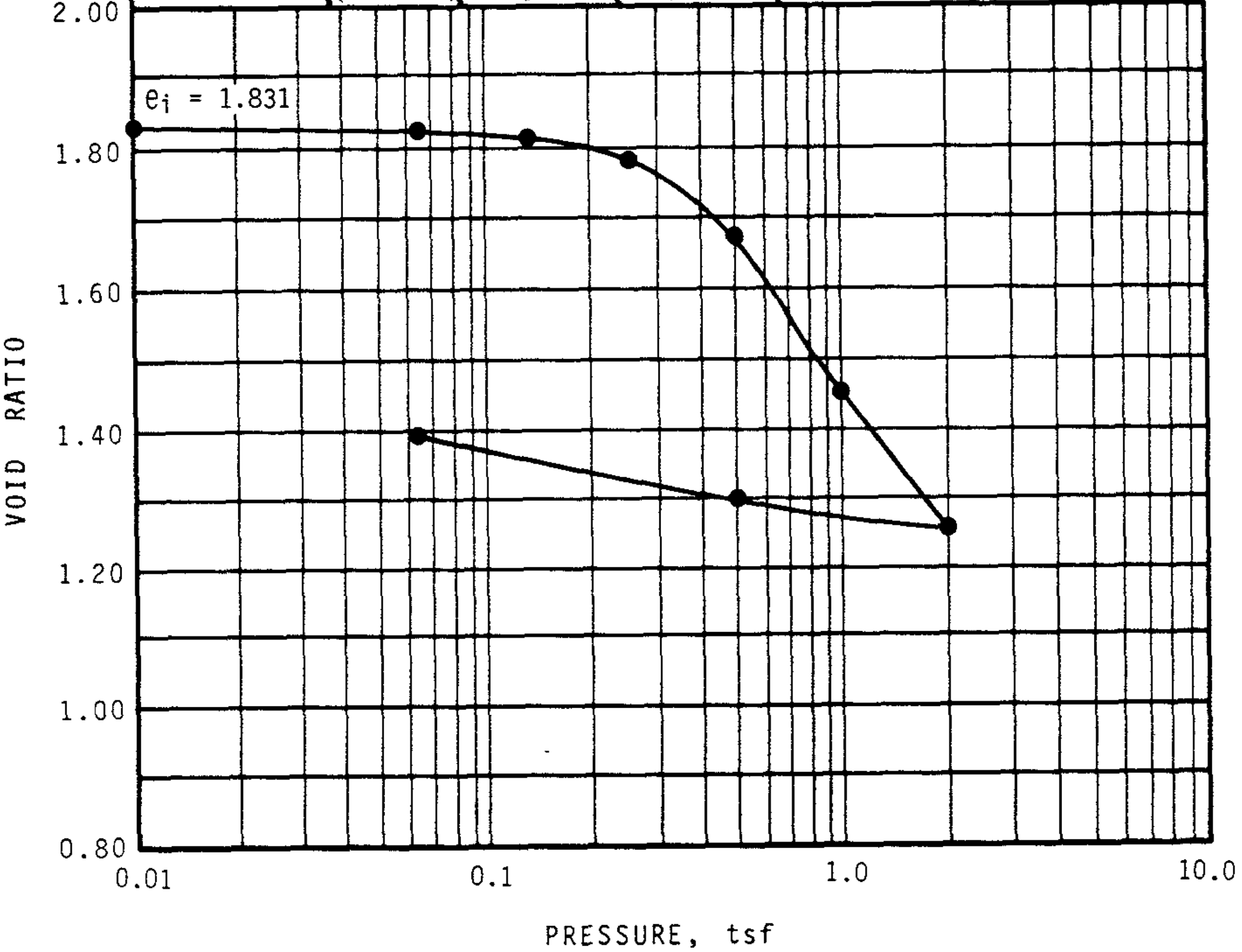
TYPE OF SPECIMEN: 3-IN. DIAMETER SHELBY TUBE

TYPE OF TEST: UNCONSOLIDATED-UNDRAINED

SPECIMEN AND SYMBOL	INITIAL MOISTURE CONTENT (%)	INITIAL DRY DENSITY (pcf)	CONFINING PRESSURE (kg/cm^2)	DEVIATOR STRESS PEAK (kg/cm^2)	PORE PRESSURE AT FAILURE
7-1 ●	55.18	67.10	0.171	0.16	—
7-2 ▲	65.28	60.16	0.405	0.22	—
7-3 ■	74.08	55.57	0.732	0.19	—



SAMPLE NO. 7 - 2	SUMMARY OF TEST RESULTS					
	SPECIFIC GRAVITY	MOISTURE CONTENT, (%)	DRY DENSITY, (PCF)	PERCENT OF SATURATION, (%)	HEIGHT (IN.)	DIAMETER (IN.)
INITIAL	2.84	64.4	62.6	99.9	0.9580	2.80
FINAL	(Assumed)	49.0	74.1	100.0	0.8094	



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CONSOLIDATION TEST RESULTS
COYOTE CREEK IMPROVEMENTS

Figure A-52

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GRAIN SIZE DISTRIBUTION CURVES

COYOTE CREEK IMPROVEMENTS

BORING NO.	SAMPLE NO.	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
1	2	8'	———————	—	—	(SM) Silty Sand
1	3	12½'	—— —— ——	—	—	(SM) Silty Sand
10	1	2½'	– – – – – –	—	—	(SM) Silty Sand w/ some Gravel
13	2	5'	···············	—	—	(ML) Sandy Silt w/ Clay Seams

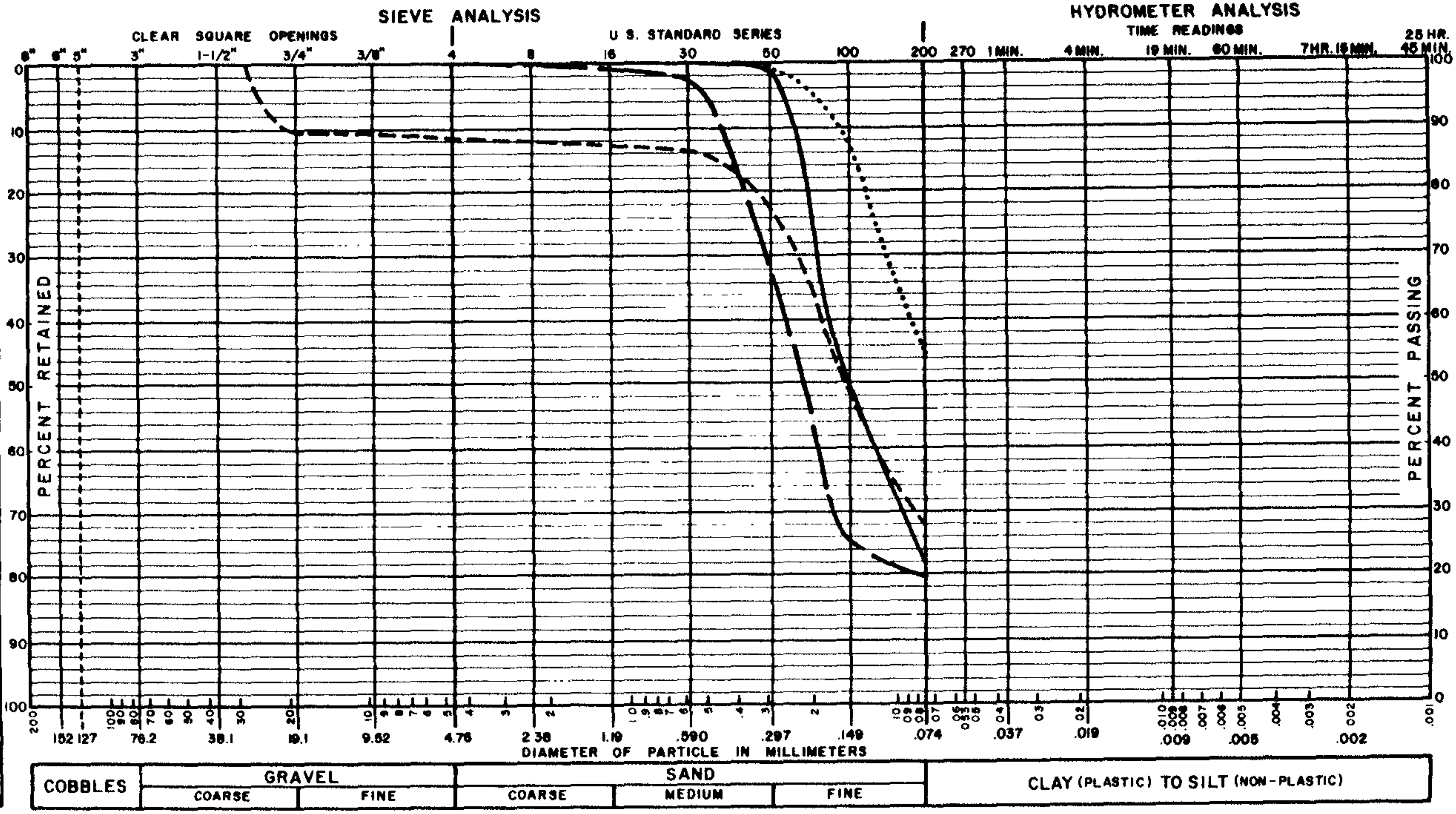

Figure A-53

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GRAIN SIZE DISTRIBUTION CURVES

COYOTE CREEK IMPROVEMENTS

BORING NO.	SAMPLE NO.	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
16	2	5½'	———	—	—	(SM) Silty Sand
17	5	14½'	— — —	—	—	(SM) Silty Sand
18	2	4½'	- - - -	—	—	(ML) Sandy Silt

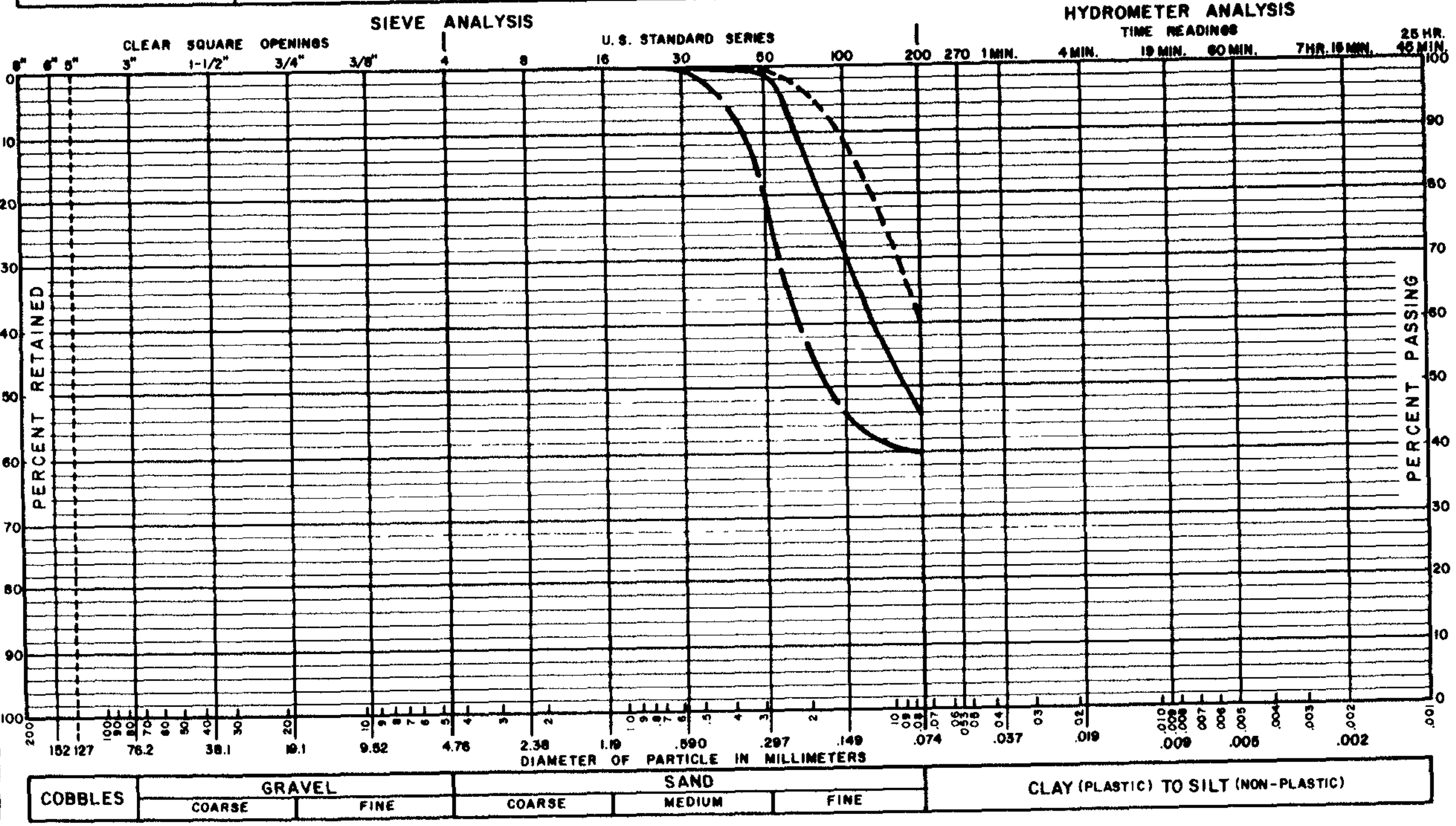

Figure A-54

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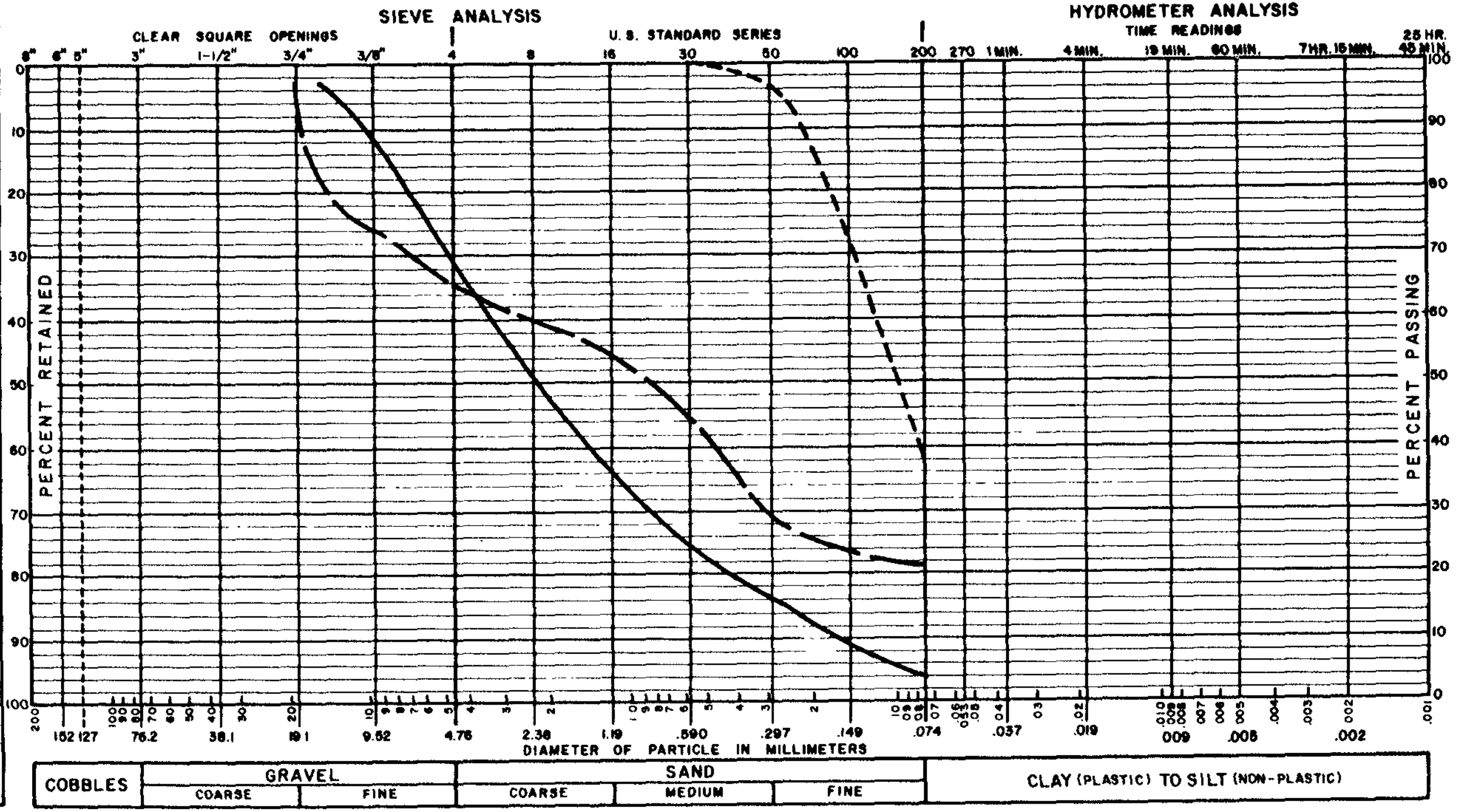
BORING NO.	SAMPLE NO.	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
22	2	5'	———	—	—	(SW) Gravelly Sand
22	3	7½'	— — —	—	—	(SM) Gravelly Sand
23	1	3'	- - - -	—	—	(SM) Silty Sand



GRAIN SIZE DISTRIBUTION CURVES

COYOTE CREEK IMPROVEMENTS

Figure A-55

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GRAIN SIZE DISTRIBUTION CURVES

COYOTE CREEK IMPROVEMENTS

BORING NO.	SAMPLE NO.	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
23	5	11½'	———	—	—	(ML) Sandy Silt
24	2	5'	— — —	—	—	(SM) Silty Sand
25	5	15½'	- - - -	—	—	(SM) Silty Sand w/ Fine Gravel

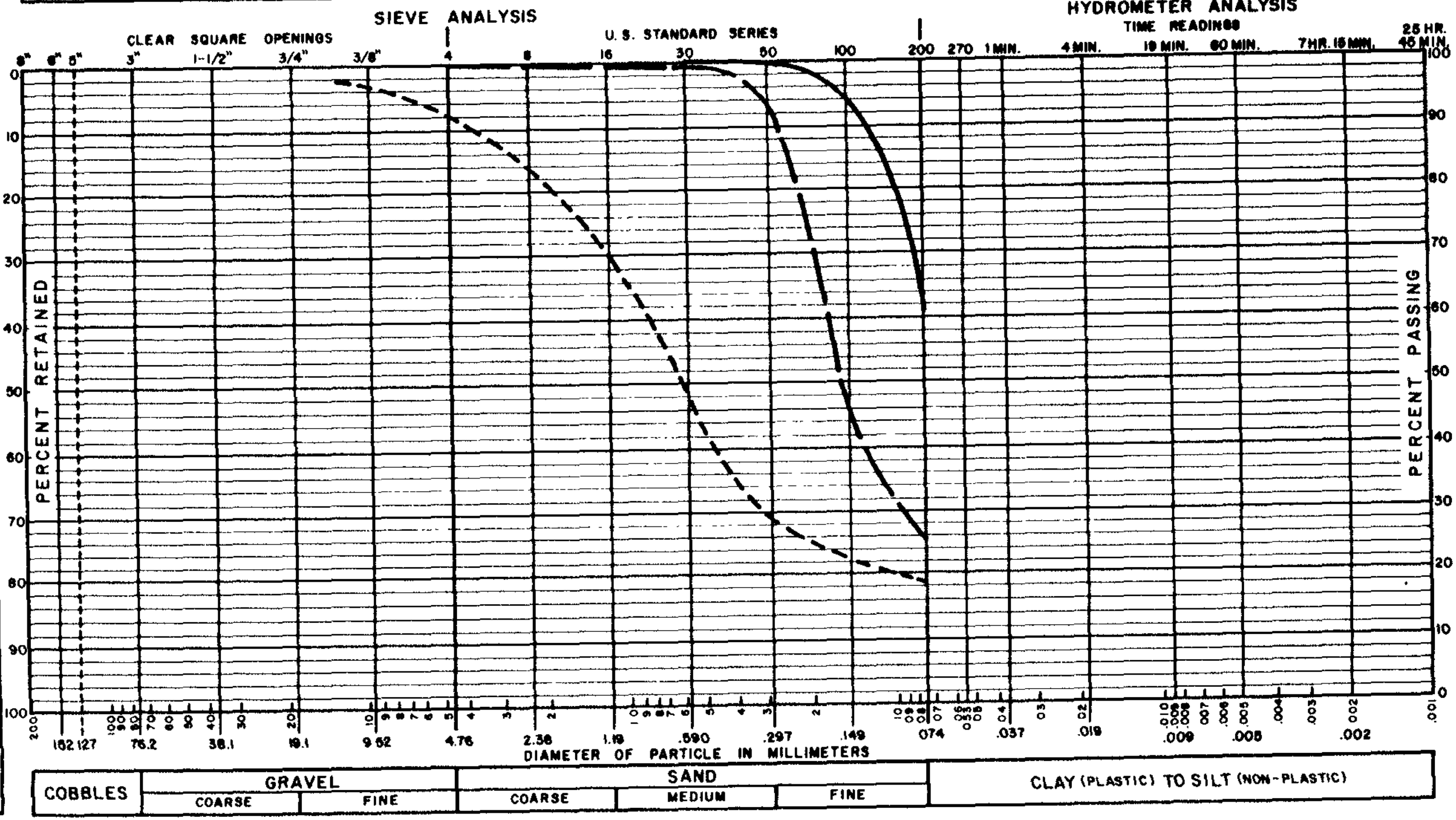

Figure A-56

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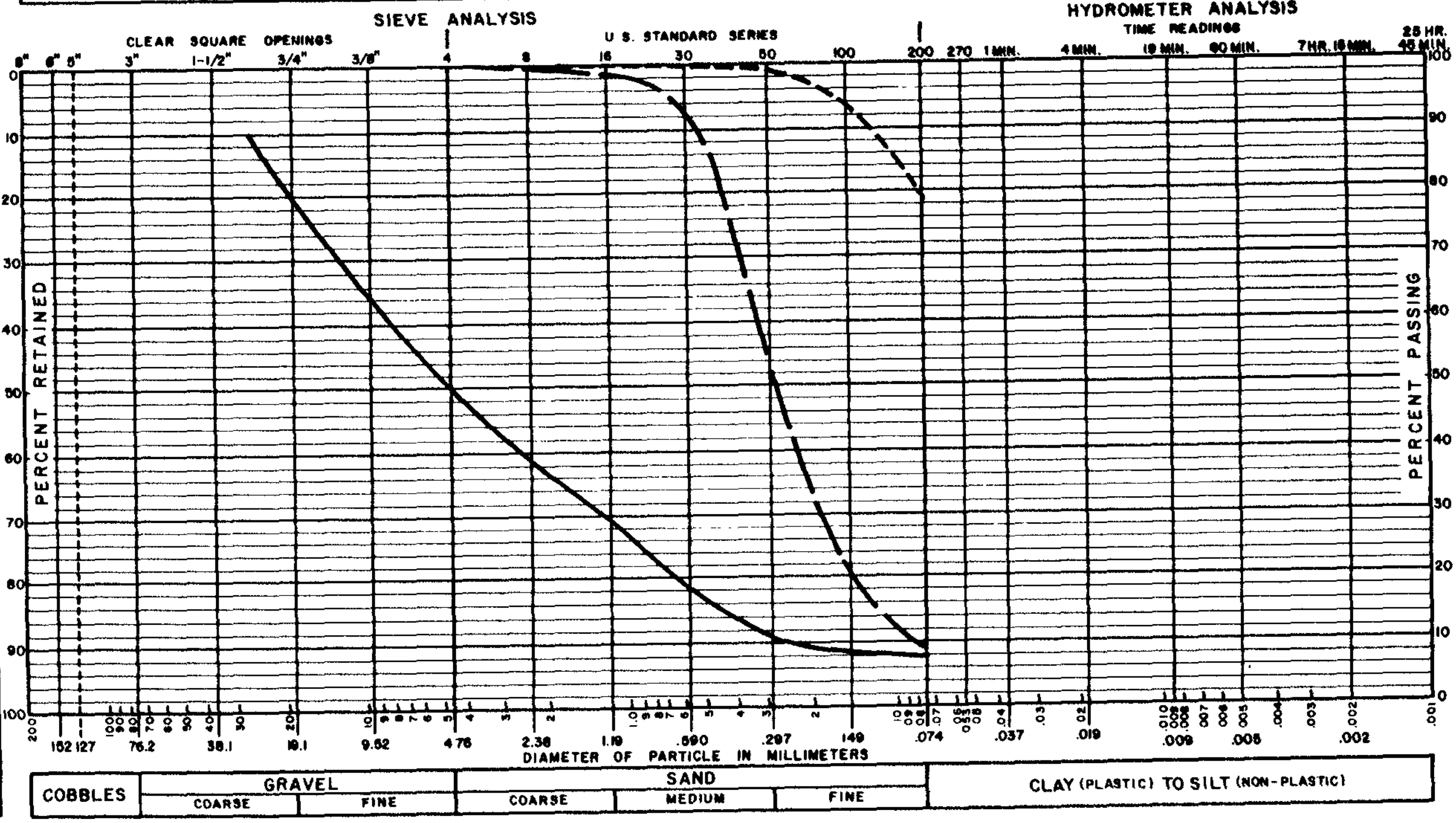
GRAIN SIZE DISTRIBUTION CURVES

COYOTE CREEK IMPROVEMENTS

Figure A-57

BORING NO.	SAMPLE NO.	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
25	7	20'	———	—	—	(GP-GM) Sandy Gravel
26	5	11½'	— — —	—	—	(SP-SM) Sand
27	2	5½'	- - - -	30	10	(CL) Sandy Clay



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Project No. 15945W

GRAIN SIZE DISTRIBUTION CURVES

COYOTE CREEK IMPROVEMENTS

BORING NO.	SAMPLE NO.	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
27	6	20½'	———	—	—	(SM) Silty Sand
28	4	8½'	— — —	—	—	(SC-SM) Clayey Sand
28	5	11'	- - - -	—	—	(SM) Silty Sand

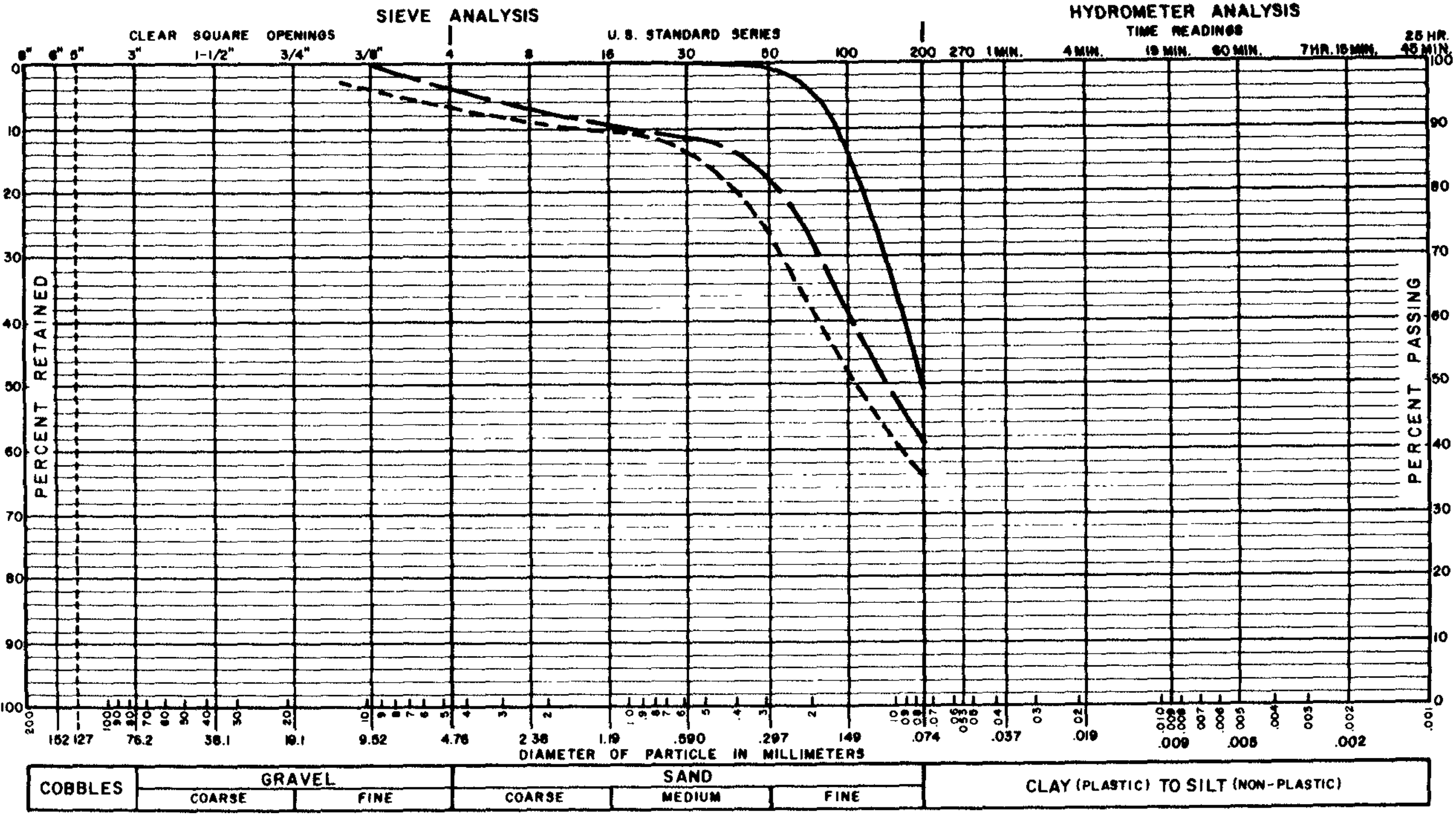
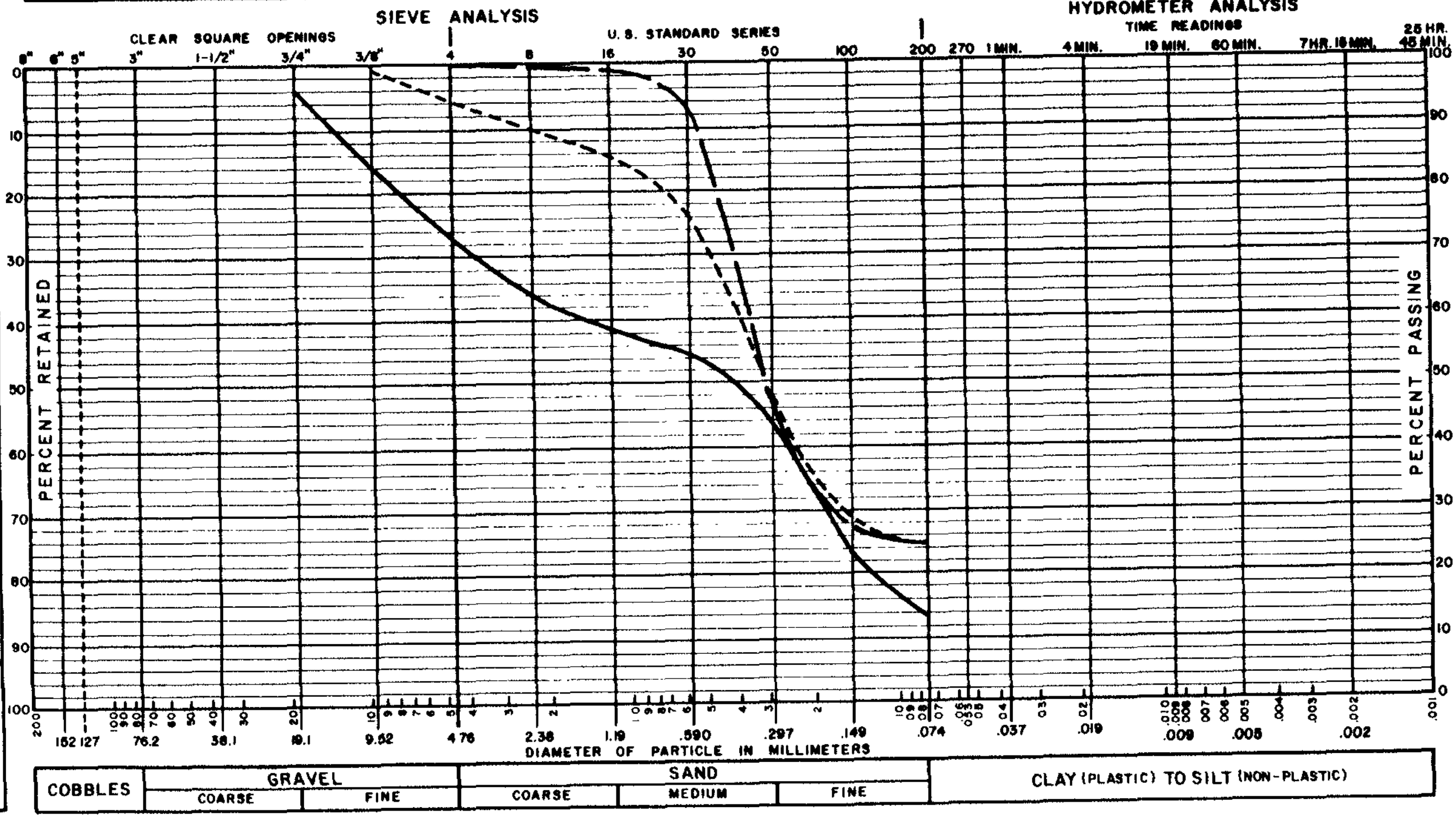


Figure A-58

GRAIN SIZE DISTRIBUTION CURVES

COYOTE CREEK IMPROVEMENTS

BORING NO.	SAMPLE NO.	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
28	7	21½'	———	—	—	(SM) Gravelly Sand
29	4	9'	— — —	—	—	(SM) Silty Sand
29	6	15'	- - - -	—	—	(SM) Silty Sand



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Project No. 15945W

Figure A-59

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BORING NO.	SAMPLE NO.	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
30	1	2½'	———	30	10	(CL) Silty Clay w/ some fine Sand
31	2	4½'	— — —	—	—	(ML) Sandy Silt

GRAIN SIZE DISTRIBUTION CURVES

COYOTE CREEK IMPROVEMENTS

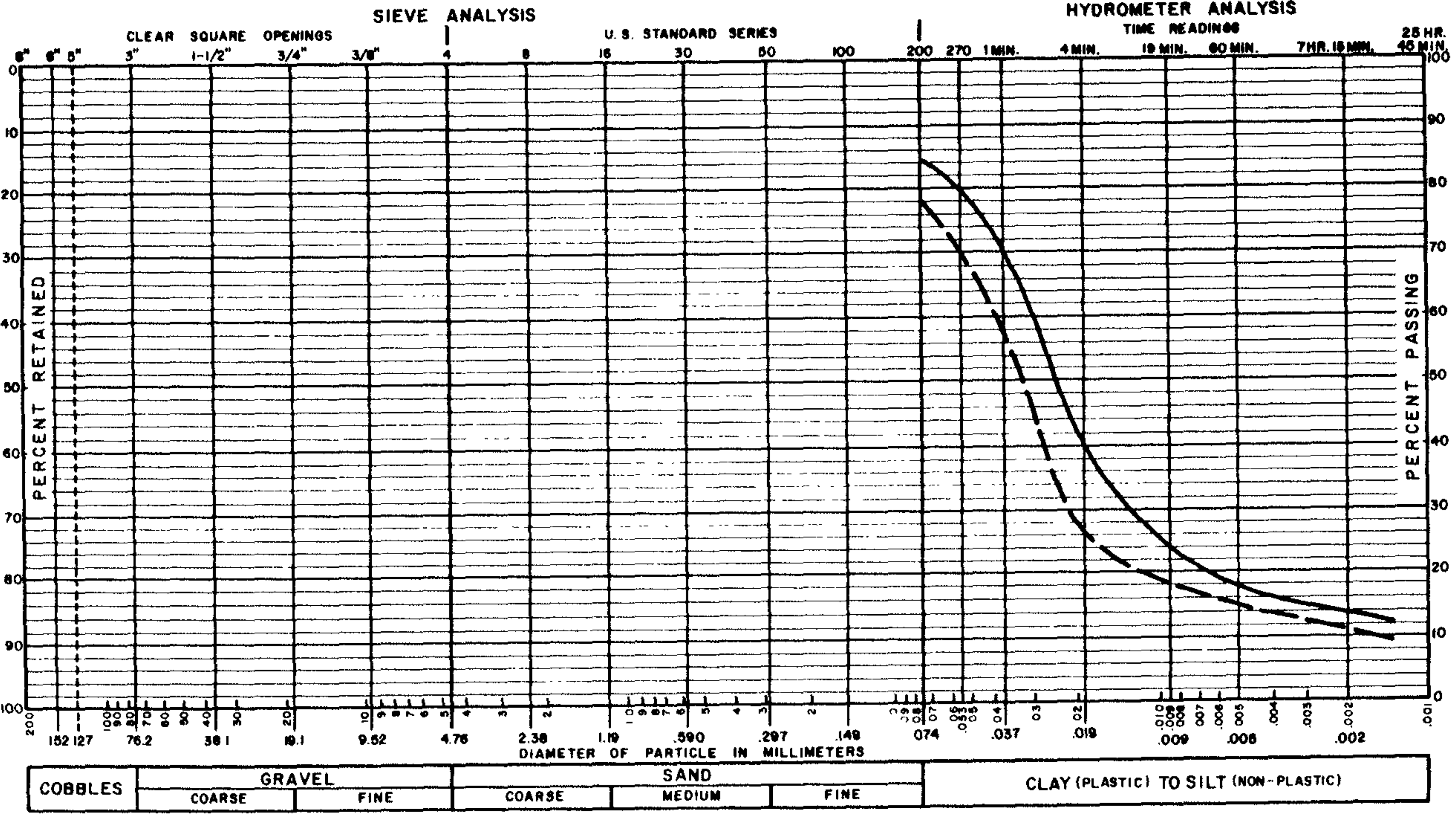

Figure A-60

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GRAIN SIZE DISTRIBUTION CURVES

COYOTE CREEK IMPROVEMENTS

BORING NO.	SAMPLE NO.	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
P1	4	19'	———————	—	—	(SW-SM) Gravelly Sand
P6	1	4'	···············	—	—	(ML-CL) Clayey Silt w/fine Sand
P6	2	9½'	—— —— ——	—	—	(SM) Silty Sand
P6	5	25'	– – – – – –	—	—	(SM) Silty Sand

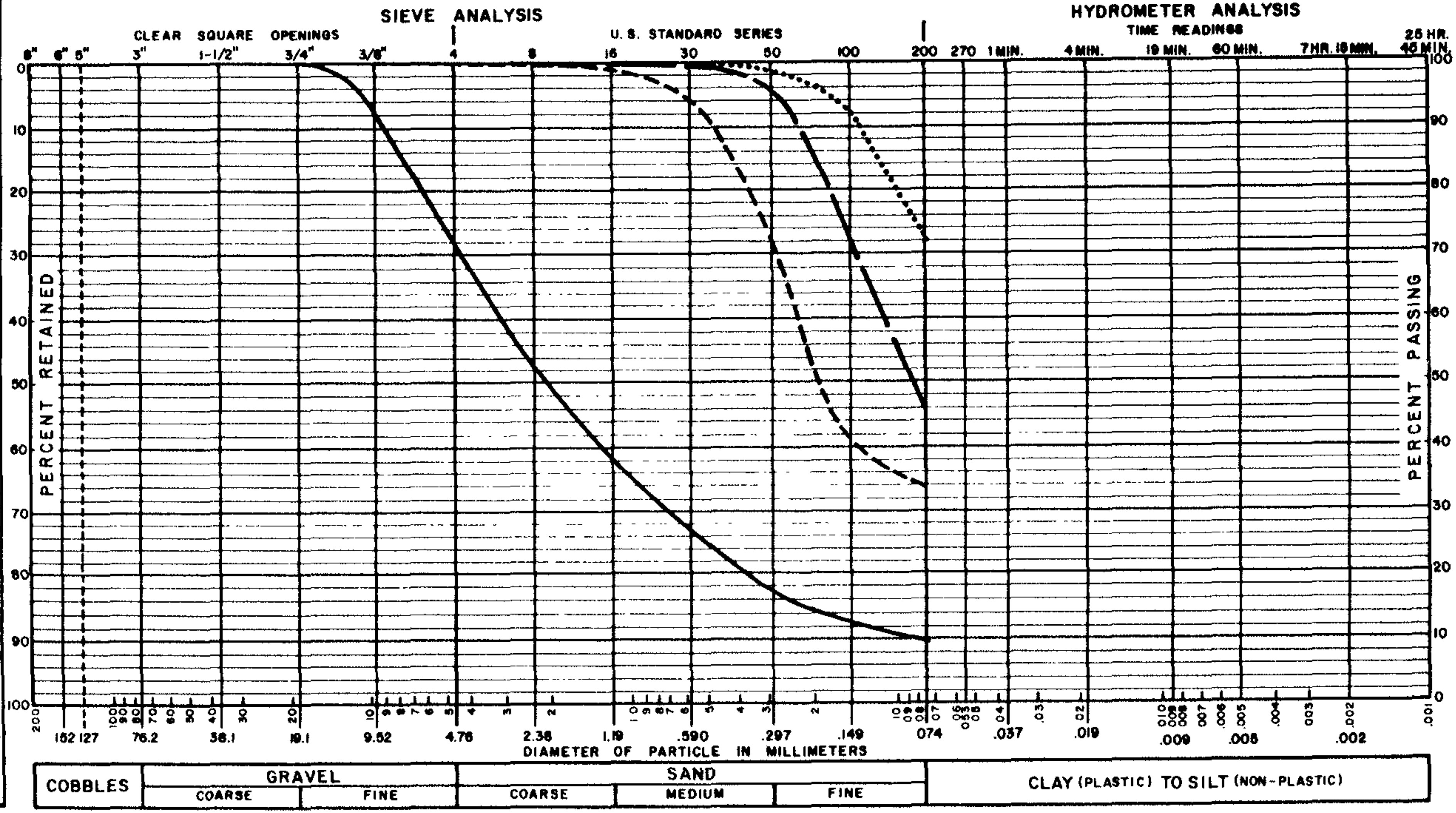


Figure A-61

Woodward-Clyde Consultants

Project No. 15945W

GRAIN SIZE DISTRIBUTION CURVES

COYOTE CREEK IMPROVEMENTS

SAMPLE DESIGNATION	DEPTH, FT.	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
Raisch Pit, Bank Run	—	———	—	—	(GC) Clayey Gravel
Gradeway Pit, Bank Run	—	— — —	—	—	(GC) Clayey Gravel
Dumbarton Quarry, Pit Run	—	- - - -	—	—	(GP-GM) Sandy Gravel

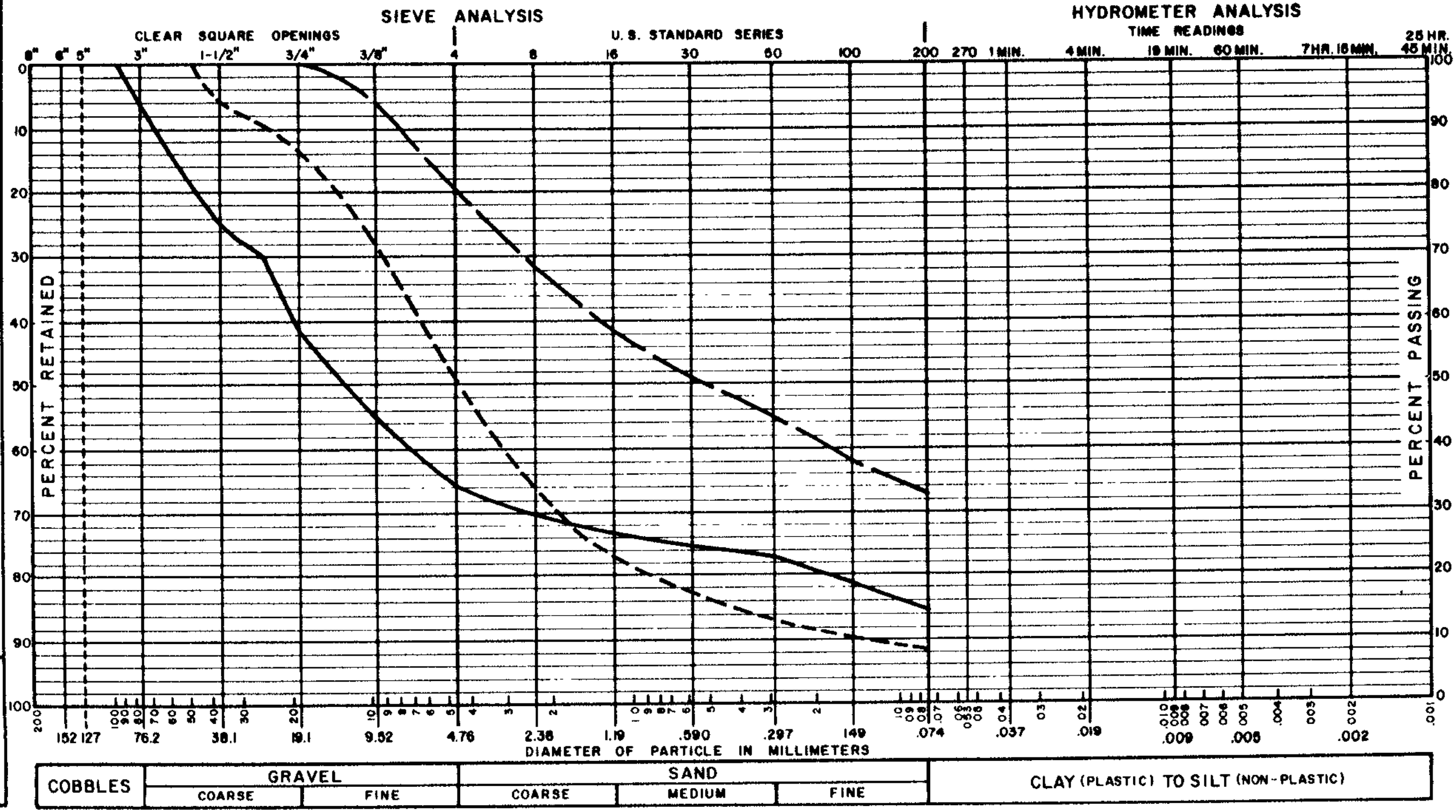
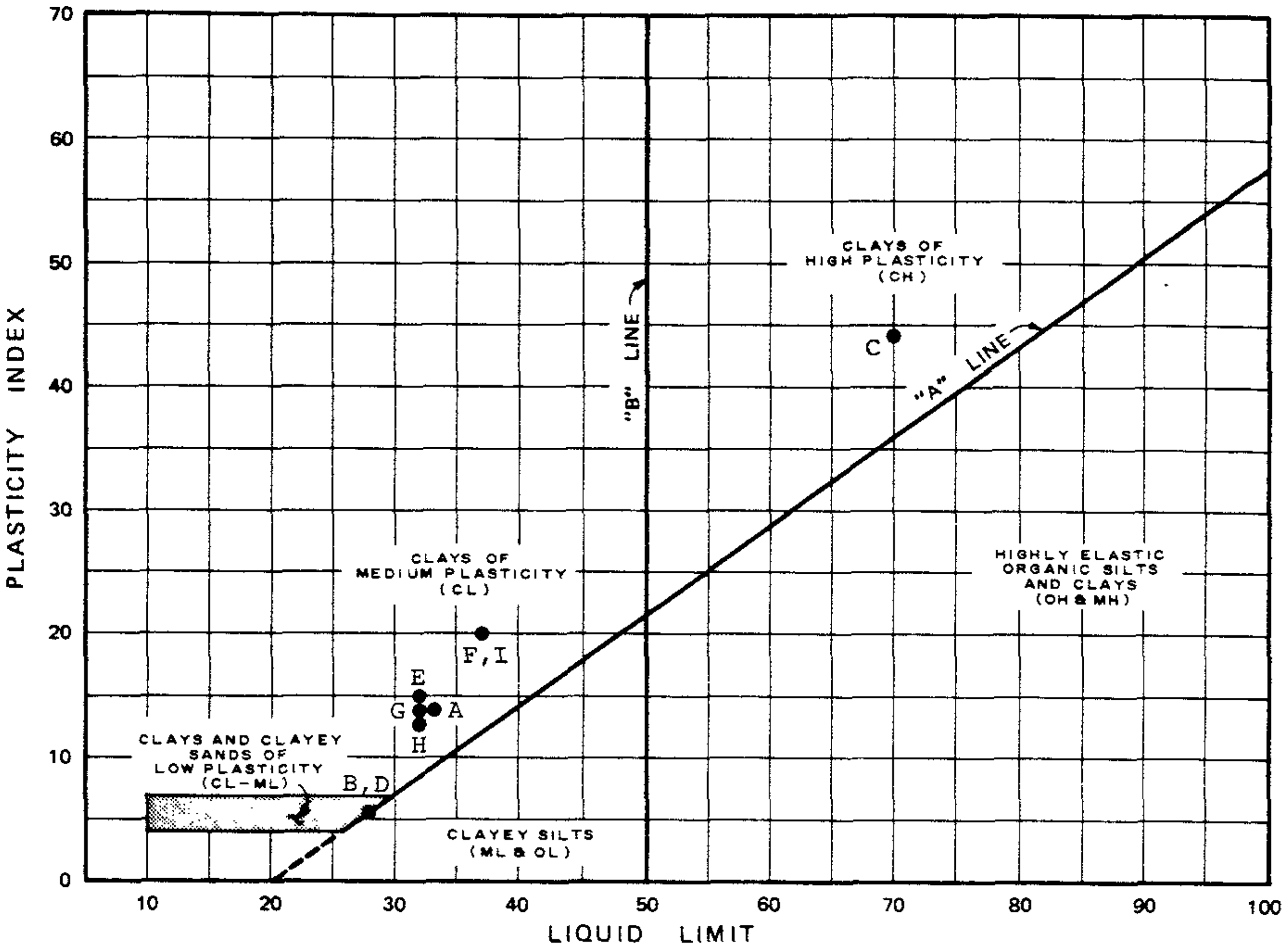


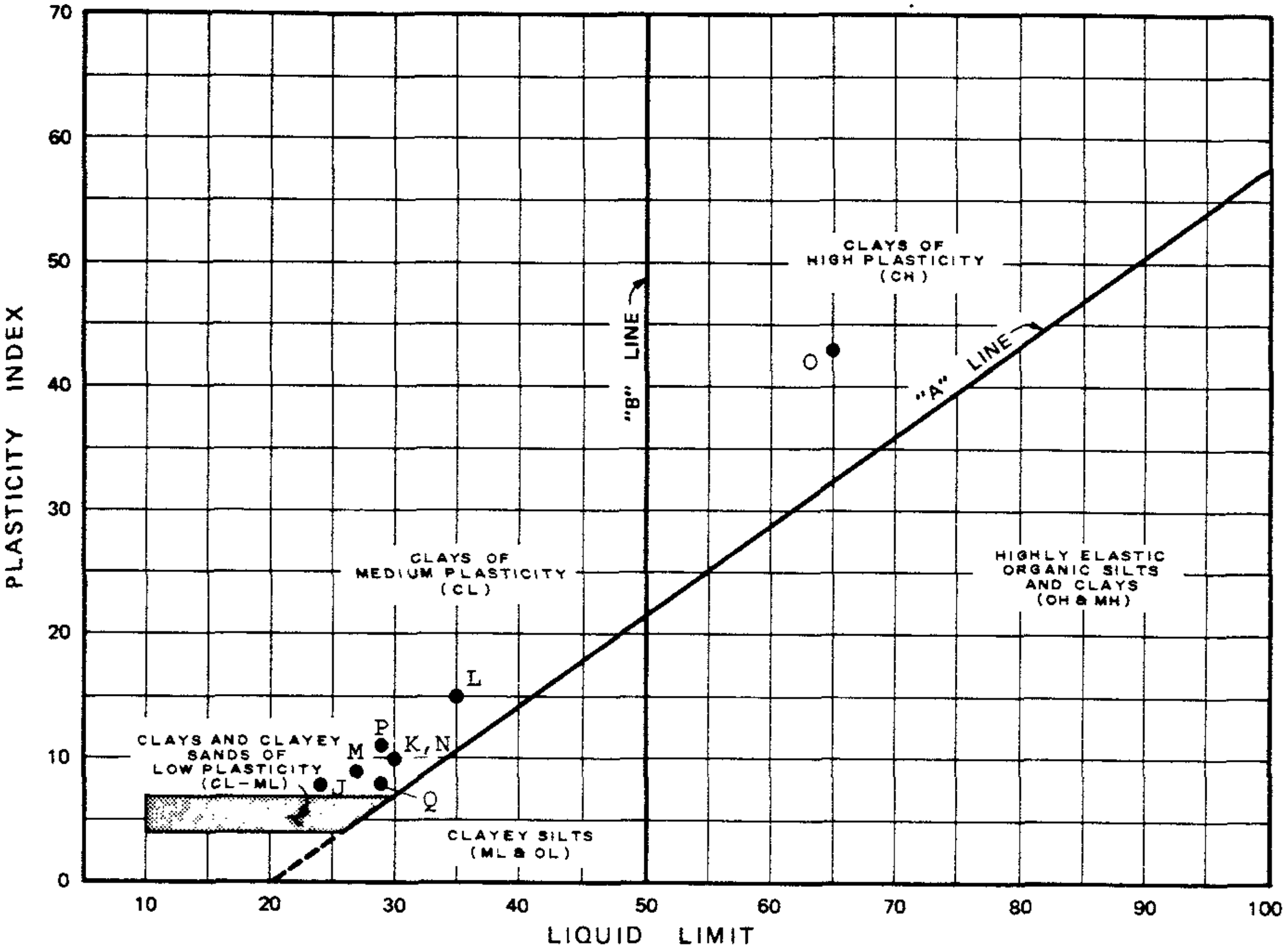
Figure A-62



CLASSIFICATION TEST RESULTS

SAMPLE IDENTIFICATION				ATTERBERG LIMITS			GRAIN SIZES - % DRY WEIGHT			
LETTER DESIG'N	BORING NO	SAMPLE NO.	DEPTH, FT.	LIQUID LIMIT	PLASTICITY INDEX	PLASTIC LIMIT	SAND	SILT	CLAY	COLLOIDAL
A	P3	1	2'	33	14	19				
B	4	2	8½'	28	6	22				
C	7	2	8'	70	44	26				
D	11	1	2'	28	6	22				
E	12	2	5'	32	15	17				
F	14	1	2'	37	20	17				
G	16	1	3'	32	14	18				
H	16	2	5'	32	13	19				
I	16	5	14½'	37	20	17				

Project No. 15945W **Woodward-Clyde Consultants**	PLASTICITY CLASSIFICATION COYOTE CREEK IMPROVEMENTS	Figure A-63



CLASSIFICATION TEST RESULTS

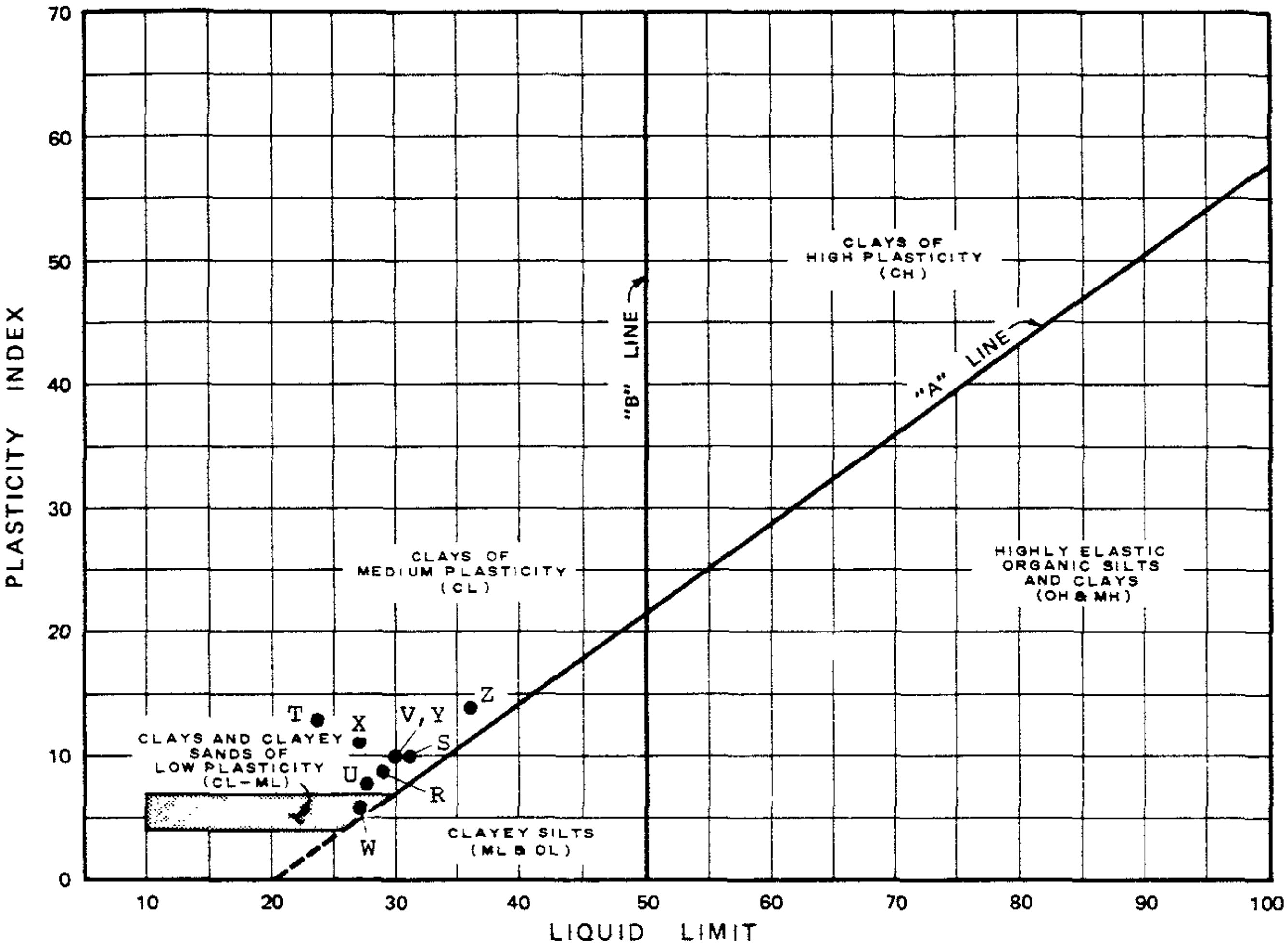
SAMPLE IDENTIFICATION				ATTERBERG LIMITS			GRAIN SIZES - % DRY WEIGHT(1)			
LETTER DESIG'N	BORING NO	SAMPLE NO	DEPTH, FT.	LIQUID LIMIT	PLASTICITY INDEX	PLASTIC LIMIT	SAND	SILT	CLAY	COLLOIDAL
J	17	1	2'	24	8	16				
K	17	2	4½'	30	10	20				
L	18	1	2'	35	15	20				
M	19	2	4½'	27	9	18				
-	20	1	3'	Non-Plastic						
N	20	2	5½'	30	10	20				
O	20	4	10'	65	43	22				
P	21	2	4'	29	11	18				
Q	22	1	3'	29	8	21				
-	23	3	8'	-	-	-	47	-	-	-

(1) Percent retained

Project No. 15945W
Woodward-Clyde Consultants

PLASTICITY CLASSIFICATION
COYOTE CREEK IMPROVEMENTS

Figure A-64

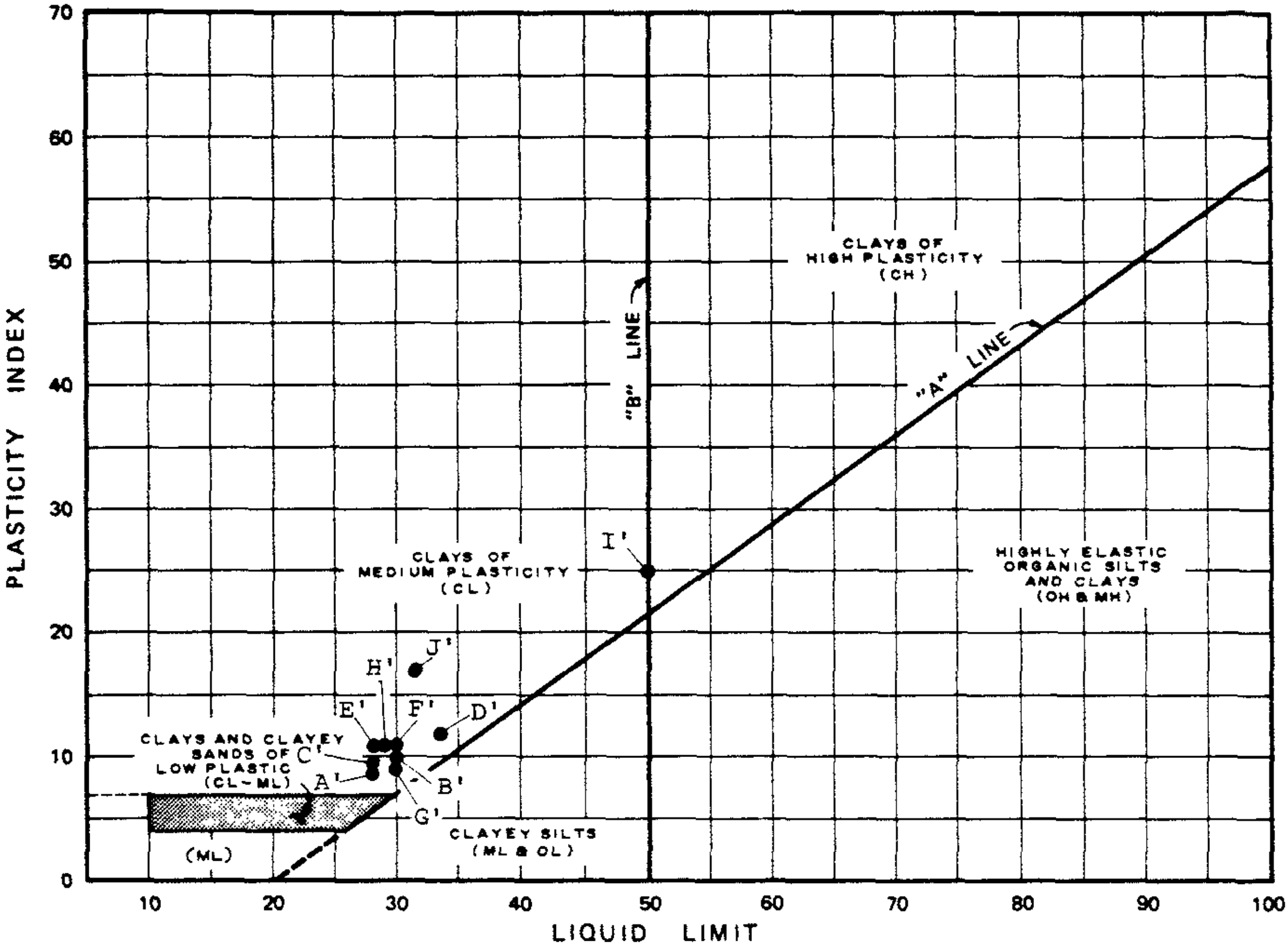


CLASSIFICATION TEST RESULTS

SAMPLE IDENTIFICATION				ATTERBERG LIMITS			GRAIN SIZES - % DRY WEIGHT(1)			
LETTER DESIG'N	BORING NO	SAMPLE NO.	DEPTH, FT	LIQUID LIMIT	PLASTICITY INDEX	PLASTIC LIMIT	SAND	SILT	CLAY	COLLOIDAL
R	24	1	2'	29	9	20				
S	25	3	7'	31	10	21				
-	25	4	9'	-	-	-	55	–	–	–
T	26	6	14½'	24	13	11				
U	27	1	3'	28	8	20				
V	27	2	5'	30	10	20				
W	27	3	7'	27	6	21				
X	28	2	4'	27	11	16	47	–	–	–
Y	30	1	2'	30	10	20	16	70	14	–
Z	30	3	6'	36	14	22				
-	31	2	4½'	-	-	-	22	67	11	–

(1) Percent retained

Project No. 15945W **Woodward-Clyde Consultants**	PLASTICITY CLASSIFICATION COYOTE CREEK IMPROVEMENTS	Figure A-65



SAMPLE IDENTIFICATION				ATTERBERG LIMITS		
LETTER DESIG'N	EXPLORATORY PIT NO	SAMPLE NO.	DEPTH, FT.	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
A'	1	1	1-2'	28	20	8
B'	2	1	½-1½'	30	20	10
C'	3	1	½-1½'	28	19	9
D'	4	1	1-1½'	33	21	12
E'	5	1	1-1½'	28	17	11
F'	6	1	0-1'	30	19	11
G'	6	3	3-4'	30	21	9
H'	7	1	0-1'	29	18	11
I'	8	1	1-2'	50	25	25
J'	10	-	Surface	32	15	17

Project No. 15945W
Woodward-Clyde Consultants

PLASTICITY CLASSIFICATION
COYOTE CREEK IMPROVEMENTS

Figure A-66

TABLE A-1

RESULTS OF LABORATORY COMPACTION TESTS

SAMPLE DESIGNATION	MATERIAL DESCRIPTION	TEST METHOD	MAXIMUM DRY DENSITY, PCF	OPTIMUM MOISTURE CONTENT % DRY WEIGHT
EP 1-1	Silty Clay (CL)	Calif. 216-F	117.8	14.1
EP 2-1	Silty Clay (CL)	Calif. 216-F	121.9	13.3
EP 3-1	Silty Clay (CL)	Calif. 216-F	127.1	10.2
EP 4-1	Silty Clay (CL)	Calif. 216-F	118.8	12.1
EP 5-1	Silty Clay (CL)	Calif. 216-F	122.4	12.7
EP 6-1	Silty Clay (CL)	Calif. 216-F	123.2	11.1
EP 6-3	Silty Clay (CL)	Calif. 216-F	118.9	13.3
EP 7-1	Sandy Clay (CL)	Calif. 216-F	132.8	10.0
EP 8-1	Silty Clay (CL-CH)	Calif. 216-F	107.4	18.7
EP 10	Silty Clay (CL) with gravel, Temporary flood control levee fill	Calif. 216-F	123.5	12.6
Raisch Pit	Clayey Gravel (GC) with cobbles, bank run	ASTM 1557 6 in mold	123.4	11.2
Grade-Way Pit	Clayey Gravel (GC) Bank run	Calif. 216-F	119.0	13.0

Project: 15945-W

Table A-2

Results of Variable Head Permeability Tests(1)

Sample Designation	Sample Type	Material Description	Permeability, k (cm/sec)
12-3-3	Drive Sample(2)	Silty Clay (CL)	9.1×10^{-8}
13-2-3	Drive Sample	Sandy Silt (ML) with clay seams	9.6×10^{-7}
16-4-3	Drive Sample	Silty Clay (CH)	8.8×10^{-9}
17-5-4	Drive Sample	Silty Sand (SM)	2.9×10^{-4}
P6-1-3	Drive Sample	Clayey Silt (ML-CL) with fine sand	1.9×10^{-6}
EP1-1	Fabricated(3)	Silty Clay (CL-ML)	1.3×10^{-6}
EP4-1	Fabricated	Silty Clay (CL)	4.4×10^{-7}
EP6-3	Fabricated	Silty Clay (CL-ML)	4.0×10^{-6}
EP7-1	Fabricated	Sandy Clay (CL) with gravel	1.4×10^{-6}
Raisch Pit	Fabricated	Clayey Gravel (GC) with cobbles	1.1×10^{-4}
Grade-Way Pit	Fabricated	Clayey Gravel (GC)	6.5×10^{-5}

NOTES:

(1) Variable head permeability tests performed using a confining pressure of 4.2 Kg/cm^2

(2) Soil samples obtained using a modified California drive sampler (2 in. inside diameter)

(3) Fabricated samples compacted to 90 percent of maximum dry density as determined by Test Method California 216-F or ASTM 1557

Project: 15945-W

TABLE A-3

SUMMARY OF FIELD VANE SHEAR TEST RESULTS

- Open Water Area

Boring No.	Approximate Depth (ft)	Approximate Elevation (ft)	Shear Strength (1) (psf)	Remolded (1) Shear Strength (psf)
1	2	+ 1/2	170	20
1	5	-2 1/2	210	80
2	2	-2	90	30
2	4 1/2	-4 1/2	150	70
3	3 1/2	-1	370	40
3	8 1/2	-6	300	120
4	3	- 1/2	240	40
4	5	-2 1/2	340	140
5	3	- 1/2	190	40
5	7	-4 1/2	210	100
6	4	-2 1/2	300	100
7	4 1/2	-2	340	100
7	9 1/2	-7	480	300

- Existing Salt Pond Levee

Boring No.	Approximate Depth (ft)	Approximate Elevation (ft)	Shear Strength (1) (psf)	Remolded (1) Shear Strength (psf)
2A	2	+7	1,490	-
2A	4	+5	590	130
2A	6	+3	670	70
2A	8	+1	450	100
2A	10	-1	510	70
2A	13	-4	480	100
2A	15 1/2	-6 1/2	450	130
2A	16	-7	650	390
3A	5 1/2	+4 1/2	670	100
3A	7 1/2	+2 1/2	300	40
3A	9 1/2	+ 1/2	560	100
3A	11 1/2	-1 1/2	480	70
3A	13	-3	480	110
3A	15	-5	630	120
3A	17	-7	450	120
3A	19	-9	590	220

TABLE A-4

SUMMARY OF QUARRY DATA

NAME	LOCATION	OPERATOR	MATERIAL DESCRIPTION	ANTICIPATED APPLICATION	ESTIMATED(1) COST
Dumbarton Quarry	Coyote Hills Newark	Dumbarton Quarry Associates P.O. Box 487 Fremont, CA 94537 Joe Evans 415-793-8862	Franciscan Formation Sandy Gravel (GP-GM) 2 in minus, non-plastic	Below water portion of displacement or floating embankment in salt pond; bridging material	$1.50/Ton - pit run $1.95/Ton - hauling
Sand Pit	Sabercat Road Fremont	Oliver DeSilva 22991 Clawiter Rd Hayward, CA Joe Birch 415-783-9220	Santa Clara Formation or Miocene(?) Sandstone Decomposed to clayey sand-sandy clay (SC-CL) with some rock fragments	General Levee Fill	$1.30/Ton (loaded into truck) hauling cost not provided
Curtner Pit	Scott Creek Road Milpitas	ConExCo, Inc. 12 N. Sunset Ave. San Jose, CA Tom Collins 408-258-2875	Monterey Formation Shale, weathered to fresh; ranging from angular rock fragments to Silty Clay (CL-CH) also, Briones Formation Sandstone in a clayey matrix	General levee fill	$1.25/Ton - 6 in minus $0.90/Ton - over size $1.20/Ton - hauling
Raisch Pit	Old Calaveras Road Milpitas	Dastrup Paving Co. 699 E. Brokaw Road San Jose, CA Gerry Dastrup 408-293-0733	Briones Formation Sandstone in a clayey matrix; clayey gravel (GC-GP), 4 to 6 in minus	Below water portion of floating embankment or displacement fill in salt pond; bridging material; materials with higher clay content might be used as genral levee fill	$1.00/Ton - bank run $1.12.Ton - hauling

TABLE A-4
(continued)

NAME	LOCATION	OPERATOR	MATERIAL DESCRIPTION	ANTICIPATED APPLICATION	ESTIMATED(1) COST
Grade-Way Pit	Calaveras Rd Milpitas	Gradeway Const. Company 43801 Osgood Road Fremont, CA 94539 Bill Gates	Briones Formation Sandstone in a clayey matrix; clayey gravel (GC), 3/4 in minus	General levee fill cleaner, coarser quarry materials might be used for bridging material, below water portion of floating embankment or displacement fill in salt pond	$1.00-1.10/Ton - bank run $1.50-1.75/Ton - hauling

NOTES: (1) All costs are tentative estimates, (Fall 1983) based on a minimum of 100,000 cubic yards.

APPENDIX B

SANTA CLARA VALLEY WATER DISTRICT

COYOTE CREEK ENGINEER'S REPORT
on
Interim Flood Control Improvements from San Francisco Bay to Highway 237

Prepared by

Jose L. Ortiz, P.E.
Associate Civil Engineer

and

Randall R. Talley, P.E.
Division Engineer

Robert R. Smith, P.E.
Head, Project Development Branch

John T. O'Halloran
General Manager

George Korbay, P.E.
Flood Control Manager

JUNE 1983

DISTRICT BOARD OF DIRECTORS

Joe Pandit	District 1	James J. Lenihan	District 5
Patrick T. Ferraro	District 2	Sig Sanchez, Chairman	At Large
Robert W. Gross, Vice Chairman	District 3	Audrey H. Fisher	At Large
Joseph H. Donohue	District 4		

Introduction

This report presents the results of an investigation of the disastrous flooding which occurred from Coyote Creek north of Highway 237 in the March 1983 storms.

Background

Coyote Creek northerly of Highway 237 has the capacity to contain less than 2000 cubic feet per second (cfs). The flow in March 1983 was an estimated 5400 cfs, causing the overtopping and subsequent erosion of the levees between Highway 237 and Dixon Landing Road. Floodwaters covered the area between Guadalupe River and Lower Penitencia Creek and from Highway 237 north to the Fremont airport (See Figure 1).

Flooding around the San Jose Sewage Treatment Plant was reportedly within half a foot of entering the buildings. Floodwaters forced evacuation of Alviso residents and overtopped the Leslie Salt Pond levee. East of Coyote Creek floodwaters overtopped Highway 17 and flooded portions of the Milpitas Manor. Traffic on Highways 17 and 237 was blocked.

Following numerous meetings and discussions with affected property owners and District staff, an interim flood control project has been developed which would provide a greater level of protection to the Alviso and Milpitas areas and properties adjacent to Coyote Creek north of Highway 237.

Project Description

The proposed flood control measures would provide protection against flooding from events of comparable magnitude to that which occurred in March 1983. The proposed measures would be constructed at various sites along Coyote Creek as shown in Figure 2 and would consist of the following work:

Site #1. The existing levee that controls tidal influence and is located north of the City of San Jose's sewage treatment plant sludge ponds would be lowered approximately three feet to elevation 7.0 (NGVD) for a distance of 800 feet. Two 48-inch tide gates would be placed in the lowered levee in addition to the two existing gates to drain freshwater flooding below elevation 7.0 feet behind the levee.

Site #2. A levee would be constructed to elevation 10.0 feet across the drainage channel between the salt pond levee and the sludge pond levee. A 48-inch outfall with flap gates would be placed in the proposed levee to accommodate drainage from the treatment plant and to prevent any tidal influence in the sludge pond area.

Site #3. A levee would be constructed to elevation 10.0 feet on private property around the Kingsford Company charcoal operation and would conform to the access road adjacent to the east. The City of San Jose's chain link fence securing the bomb disposal site and flammable storage building would be relocated adjacent to the proposed outside toe of this levee.

Site #4. The existing roadway would be lowered to elevation 7.0 feet for a distance of 450 feet.

Site #5. During the March 1983 flooding, most of the ramp betweeen the top of sludge pond levee and the access road was washed away. The remainder of the ramp would be removed to allow overflows to the west of the creek to reach the outfall weir at Site #1.

Site #6. A levee would be constructed to elevation 10.0 feet between the sludge pond levees. Existing vehicular access would be maintained over the new levee by providing appropriate ramps or road grades.

Site #7. Debris and selected vegetation will be removed between the west bank of the creek and the sludge pond levee to increase the discharge rating of the constricted section of the creek. No mature riparian vegetation would be removed.

Site #8. A levee would be constructed to elevation 13.0 feet between the sludge pond levee and the creek levee. Existing vehicular access would be maintained over the new levee by providing appropriate ramps.

Site #9. Material is proposed to be taken from the area west of the creek to construct the levee at Site #8. The excess material from Sites #1, #4 and #5 will be used to construct the proposed levees at Sites #2, #3 and #6.

Site #10. An eight-inch high asphalt dike or concrete curb would be constructed along the existing access road to the City of Milpitas' sewage pumping station for approximately 2,300 feet in the section parallel to Highway 17. This would prevent floodwaters from crossing over Highway 17 into Milpitas Manor. At the entrance to the pumping station, about 160 feet of the section of the road running east and west will be lowered one foot. This will provide an outlet to the north for floodwaters on the property east of the creek. At the southerly end of the proposed dike or curb, an earth berm will be constructed to elevation 13.0 feet across the Highway 17 roadside ditch and will confrom to the access road and the highway. A 36-inch CMP will be placed in the berm to accommodate highway drainage.

Site #11. The east bank levees would be repaired to the condition prior to the March 1983 flooding.

Right of entry permits to accommodate the proposed interim flood control measures have been secured from the Cities of San Jose and Milpitas and from the owner of the property where the charcoal operation is situated. A right of entry would also be secured from the owner of the property between the creek and Highway 17. These permits shall remain in effect for a period of four years from the date of execution.

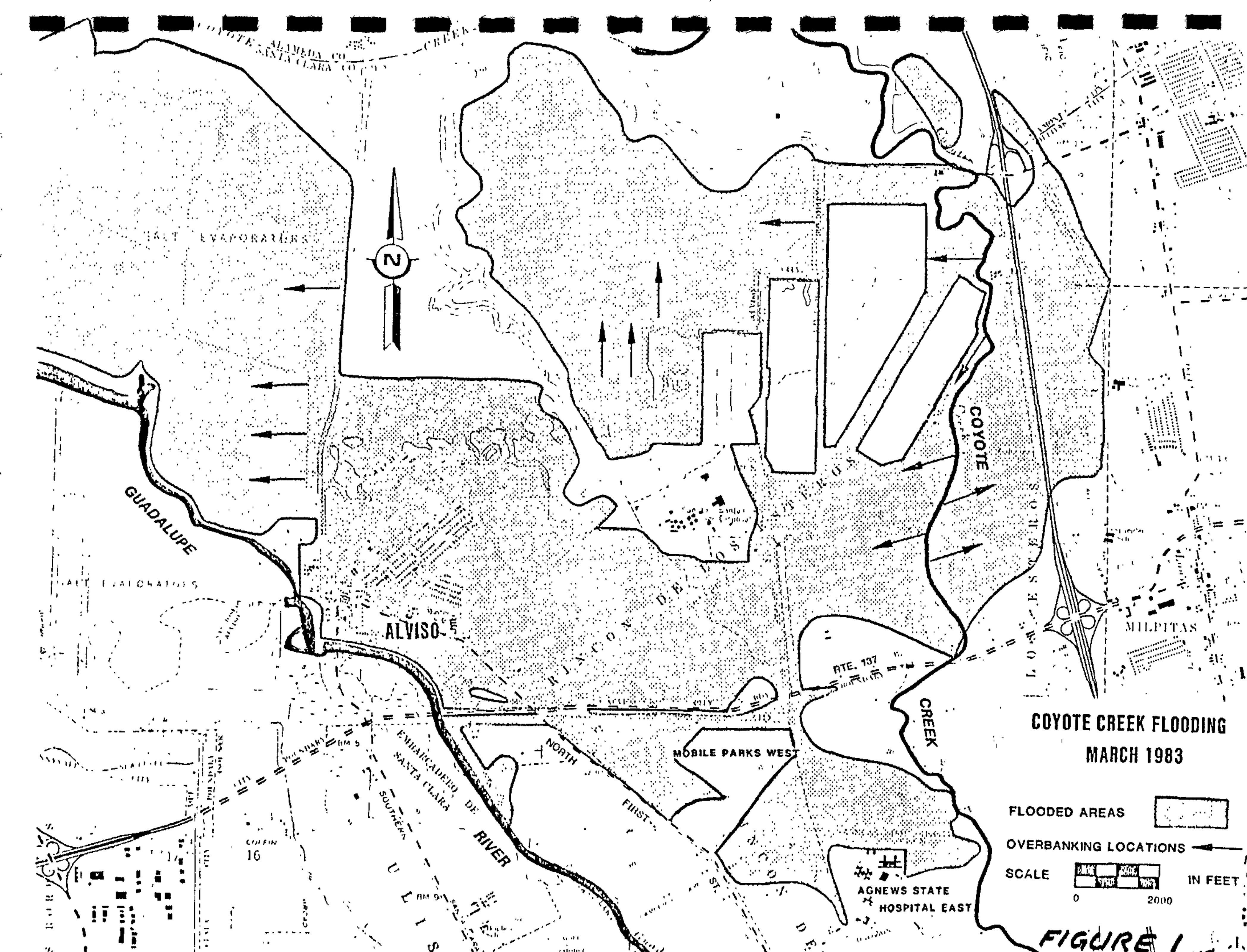
The estimated project cost is $300,000 and has been included in the East Zone Maintenance Program of the 1983-84 budget. The proposed work would be completed by October 1983.

It is recommended that the measures proposed in this report be approved and construction plans be prepared for implementation of the project.

Engineering Approval:

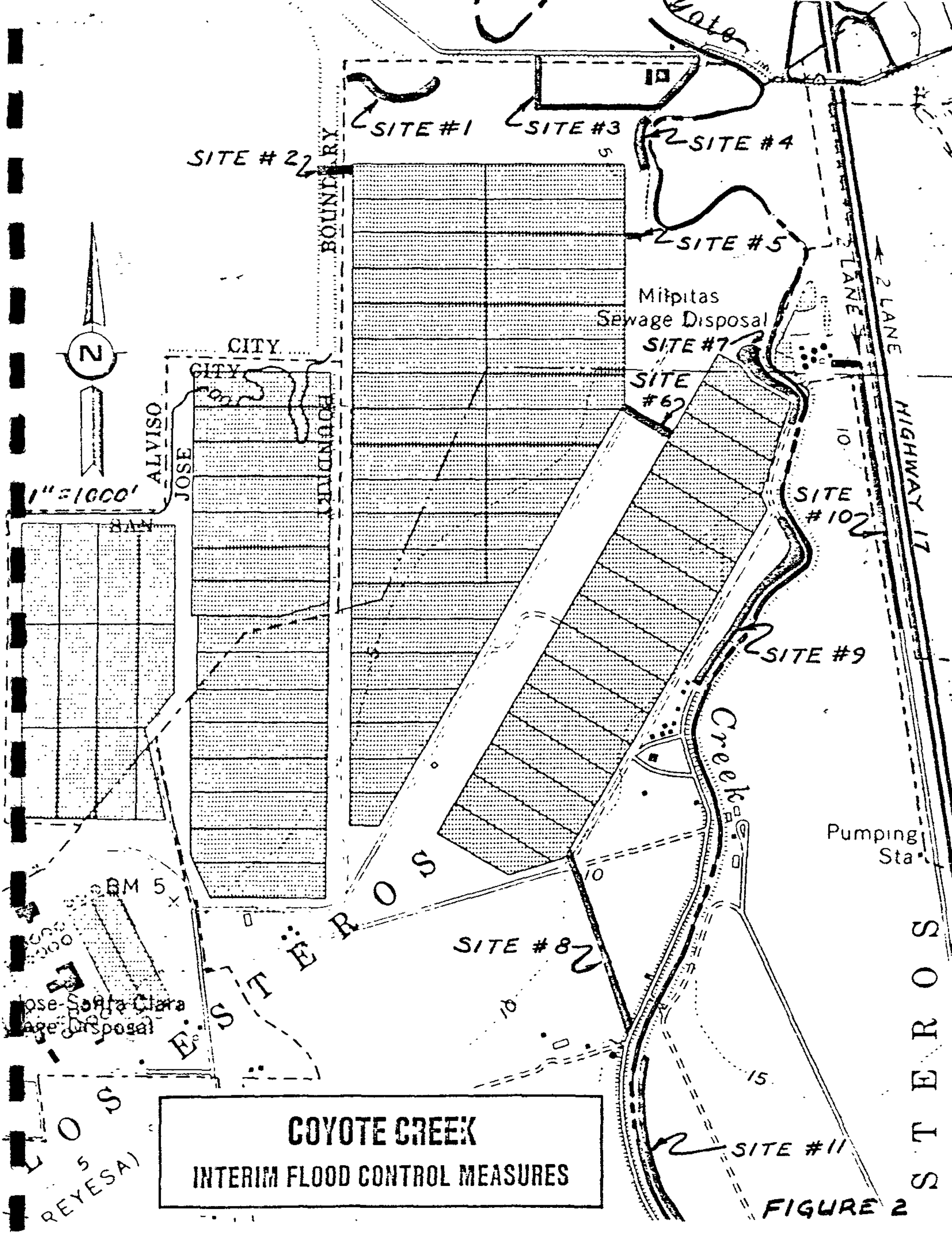
George Korbay

George Korbay, RCE No. 13462
Flood Control Manager



COYOTE CREEK FLOODING
MARCH 1983

FIGURE 1



COYOTE CREEK

INTERIM FLOOD CONTROL MEASURES

FIGURE 2

INQUIP
ASSOCIATES, INC.

REPLY TO
☐ P.O BOX 2182
SANTA BARBARA, CA 93120
(805) 963-6785
☐

January 25, 1984

Mr. Paul Boddie
Woodward-Clyde Consultants
1180 Coleman Ave.
San Jose, CA 95110

RE: Design of Slurry Trench Cutoff Wall
for Santa Clara Water District

Dear Mr. Boddie:

Enclosed please find our literature on the earthfilled (soil-bentonite) and cement-bentonite slurry trench cutoff walls.

Should you have any additional questions, please do not hesitate to contact us.

RECEIVED
JAN 27 1984
Woodward-Clyde Consultants

Yours very truly,
INQUIP ASSOCIATES, INC.

John F. Perrizo

John F. Perrizo
Vice President

JFP:nob

enclosures

GROUNDWATER AND SEEPAGE CONTROL • SPECIALTY CONTRACTORS • SLURRY TRENCH CUTOFF WALLS • FLEXIBLE LINERS

RECEIVED
JAN 27 1984
Woodward-Clyde Consultants

INQUIP ASSOCIATES, INC
REFERENCE LIST OF RECENT SLURRY TRENCH PROJECTS

PROJECT	DESCRIPTION	OWNER	ENGINEER	GENERAL CONTRACTOR
Sludge Holding Pond Catawba, SC 1977	S-B Cutoff Wall L=3800' W=3' D=24' $129,000	Bowater Carolina Corp Catawba, SC	J. E. Sirrine Co Box 5456 Sta B Greenville, SC (803)298-6000	Randall Construction Co 124 Workman Rockhill, SC (803)327-3113
Eastmain Dam James Bay, PQ 1977-78	S-B Cutoff Wall L=6500' W=5' D=70' $150,000 (Services)	Hydro Quebec Montreal, PQ	Hydro Quebec Montreal, PQ (514)875-4311	KBR Constructors, Ltd 1183 Finch Ave West Downsview, Ontario (416)636-8657
Page Park Dam Rockford, IL 1979	S-B Cutoff Wall L=1700' W=4' D=45' $203,000	USCE Rock Island Dist Rock Island, IL	USCE Rock Island Dist Rock Island, IL F&M Branch (309)788-6361	Rockford Blacktop Constr 600 Boylston St Rockford, IL (815)877-9561
Tailings Pond Saskatoon, Sask 1979	S-B Cutoff Wall L=18000' W=4' D=29' $70,000 (Services)	PCS Cory Mining, Ltd Saskatoon, Sask	Golder Associates, Ltd Calgary, Alta Barry Mickelborough (403)259-3413	Owner
Columbus Lock & Dam Columbus, MS 1975-79-80	S-B Cofferdam Cutoff L=5900' W=3' D=39' $750,000	USCE Mobile Dist Mobile AL	USCE Mobile Dist Mobile AL F & M Branch (205)690-2011	Arundel Atkinson Ball (JV)
Dickson Dam Innisfail, Alta 1980-81	S-B Cofferdam Cutoff L=4200' W=3' D=25' $85,000 (Services)	Alberta Environment Edmonton, Alta	Underwood Mclellan, Ltd 17007 107 Ave Edmonton, Alta Gerry Walker, PE (403)483-7722	Peter Kiewit Sons, Ltd Box 9150 Postal Sta A Edmonton, Alta J. Rasmussen, Ch Engr (403)470-3509
Premier St. Landfill Leachate Control No Vancouver, BC 1981-82	S-B Cutoff Wall L=2400' W=3' D=27' $325,000 (JV)	City of No Vancouver North Vancouver, BC	Kerr Wood Leidal, Ltd 139 West 16th Street No Vancouver, BC Ted Steele, PE (604)985-5361	Globe Excavations, Ltd 132 Riverside Dr No Vancouver, BC C. Intile, Pres (604)929-1255

8-28-83

REFERENCE LIST OF RECENT SLURRY TRENCH PROJECTS (CONT'D)

PROJECT	DESCRIPTION	OWNER	ENGINEER	GENERAL CONTRACTOR
Sierra Pt. Developmt Leachate Control Brisbane, CA 1981	S-B Cutoff Wall L=1200' W=2.5' D=22' $85,000	Sierra Pt. Developmt Brisbane, CA	Woodward Clyde Consult Box 24075 Oakland, CA Virgil Baker, Sr Engr (415)444-1256	Owner
Reid Gardner Unit 4 Evaporation Ponds Moapa, NV 1982	S-B Cutoff Wall L=5000' W=2.5' D=24' $198,000	Nevada Power Company Las Vegas, NV	Converse Consultants 4055 S. Spencer, # 120 Las Vegas, NV Joe Cibor, Proj Engr (702)732-2121	R. L. Helms Development Co 790 E. Lincoln Way Sparks, NV R. Morrison, Proj Mgr (702)359-1720
Advance Wastewater TP Infilt/Perc Beds Laramie, WY 1983	S-B Cutoff Wall L=5900' W=2.5' D=18' $165,000	City of Laramie Laramie, WY	ARIX 760 Horizon Dr Grand Junction, CO John Elmer, PE	Robt Dougan Constr Co 2100 S. Valentia St Denver, CO Randy Bright, Supt
(303)243-7569	(303)752-1000			

8-28-83

1180 Coleman Avenue
San Jose, California 95110-1194
408-297-9585

Woodward-Clyde Consultants

February 6, 1984

Project: 15945-W

Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118

Attention: Mr. David McIntire

Dear Dave:

ESTIMATED COSTS
ALTERNATIVE METHODS OF LEVEE CONSTRUCTION THROUGH SALT POND
COYOTE CREEK IMPROVEMENTS

During our meeting of January 26, 1984, we described three alternative levee construction alternatives that might be considered for the salt pond area. Those included: (1) removal of the existing soft bay mud by dredging prior to levee fill placement; (2) displacement of the bay mud; and (3) floating of the new levee embankment over the soft bay mud utilizing a system of lateral berms and geogrid reinforcement.

As you requested, we have prepared a summmary of our rough cost estimates. Those estimates, including our assumptions regarding material unit costs, etc., are presented on Table 1. Sketches showing the geometric considerations for the respective alternative construction methods in areas where the new levee would adjoin the existing salt pond levee and for open water areas are presented on Figures 1 through 7. These sketches have been based, in part, on topographic and levee alignment information provided by the District.

During our meeting, a question was raised regarding the names of general contracting companies with prior earthwork construction experience on bay mud sites. A partial list of contractors with whom we are acquainted is also attached to this letter.

If you have any questions, please give me a call.

Sincerely,

Paul J. Boddie

Paul J. Boddie
Senior Project Engineer

Enclosure

Consulting Engineers, Geologists
and Environmental Scientists

Offices in Other Principal Cities

Partial List of General Engineering Contractors with Experience on Bay Mud Sites

Gallager & Burke, Inc.
344 High
Oakland, CA
(415) 261-0466

Grade-Way Construction
43801 Osgood Road
Fremont, CA
(408) 263-1518

Granite Construction Co.
580 West Beach
Watsonville, CA
(408) 724-4711

O. C. Jones & Sons
1520 4th
Berkeley, CA
(415) 526-3424

Winton Jones Contractor, Inc.
1949 Arnold Industrial Way
Concord, CA
(415) 682-1870

Chet C. Smith Trucking
515 Tunnel Avenue
Brisbane, CA
(415) 468-0800

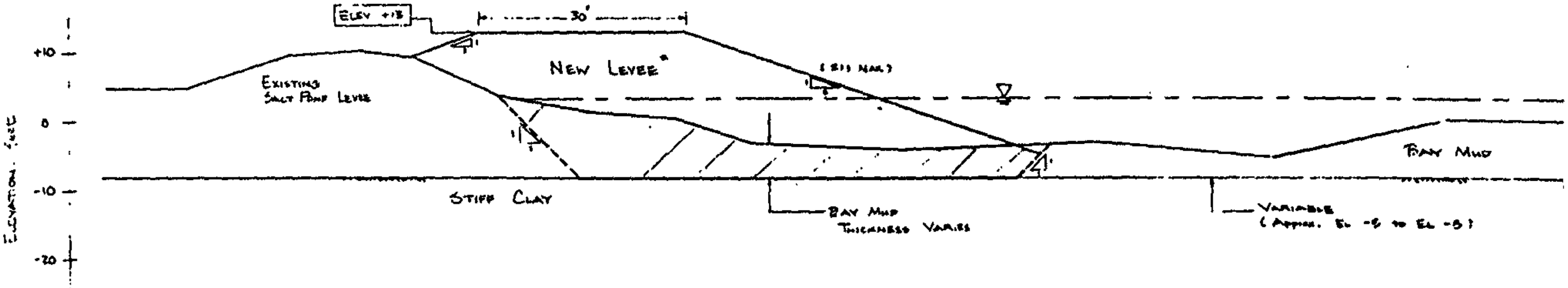
Piambo Corporation
S.F. International Airport
San Francisco, CA
(415) 588-2300

TABLE 1

ESTIMATED COSTS*
LEVEE CONSTRUCTION THROUGH SALT POND
COYOTE CREEK IMPROVEMENTS

LOCATION	ALTERNATIVE CONSTRUCTION METHOD	LEVEE SIDE SLOPE	DREDGING		GRANULAR FILL		COHESIVE FILL		GEOGRID		ESTIMATED
			Vol(yd^3)	Cost($)	Vol(yd^3)	Cost($)	Vol(yd^3)	Cost ($)	Area(yd^2)	Cost($)	Cost/Ft. of Levee
Adjoining Existing Salt Pond Levee	Dredging (Figure 1)	3:1	20	50	31	233	14	105	-	-	$390
		2:1	17 1/2	44	25	188	13	98	-	-	$330
Adjoining Existing Salt Pond Levee	Displacement (Figure 2)	3:1	-	-	31	233	14	105	-	-	$340
		2:1	-	-	25	188	13	98	-	-	$290
Adjoining Existing Salt Pond Levee	Floating Embankment (Figure 3)	3:1	-	-	26 1/2	199	14	105	14 1/2	29	$330
Open Water Areas	Dredging (Figure 4)	3:1	45	113	62	465	16	120	-	-	$700
		2:1	34	85	46	345	14	105	-	-	$540
Open Water Areas	Displacement (Figure 5)	3:1	-	-	56	417	16	120	-	-	$540
		2:1	-	-	42	315	14	105	-	-	$420
Open Water Areas	Floating Embankment to Elevation 13 (Figure 6)	3:1	-	-	30	226	16	120	22	44	$390
		2:1	-	-	29	218	14	105	19	38	$360
Open Water Areas	Floating Embankment to Elevation 9 (Figure 7)	3:1	-	-	22	164	15	113	18	36	$310

*Unit Cost Assumptions:
(1) Dredging: $2.50 per cubic yard, not including spoil disposal.
(2) Granular Fill: $7.50 per cubic yard for imported "Bank Run" or "Crusher Run" quarry material; including hauling and placement.
(3) Cohesive Fill: $7.50 per cubic yard for imported cohesive soil or soil and rock mixture; including hauling, placement and compaction.
(4) Geogrid: $2.00 per square yard for Tensar Geogrid SS2; including material handling and placement.



* Note: Below Water Portion Of New Levee Fill To Consist Of "Bank Run" or "Crusher Run" Quarry Materials. Above Water Portion To Consist Of Import or On-Site Cohesive Engineered Fill. Bank Run Materials Might Have Properties Consistent With Those Of Cohesive Fill

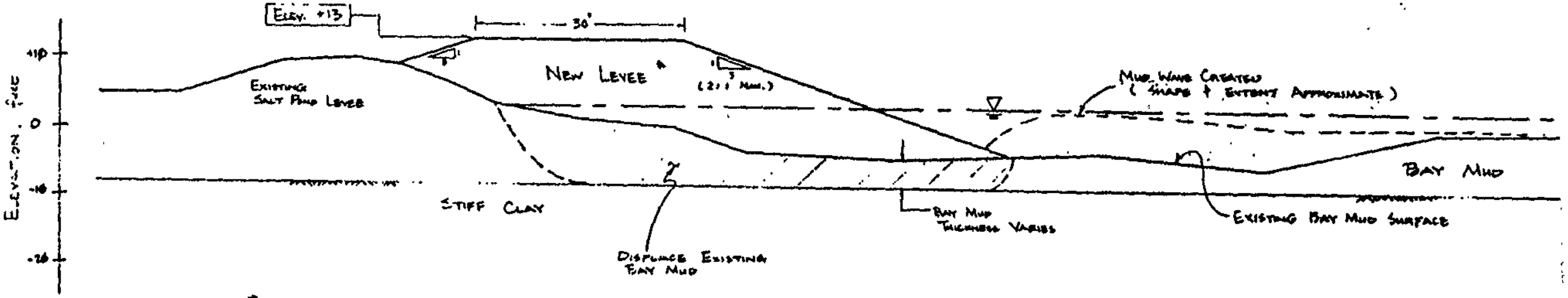
☐ : Existing Bay Mud To Be Dredged

Typical Section At Sta 79 + 30

Scale : 1 inch = 10 feet

NEW LEVEE ADJACENT TO EXISTING
SALT POND LEVEE
DREDGING ALTERNATIVE

Project No. WOODWARD-CLYDE CONSULTANTS	Figure 1



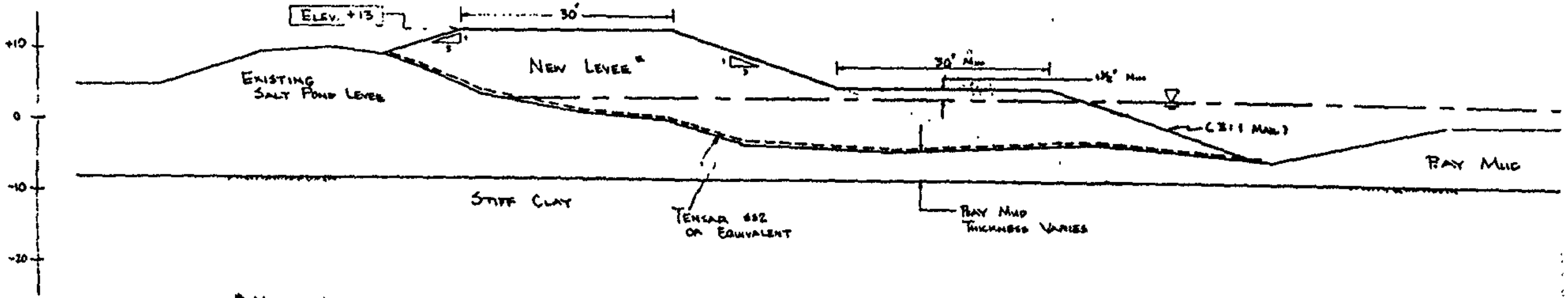
* Note: Levee Fill Materials As Described In Figure 1

Scale: 1 inch = 10 Feet

Typical Section At Sta 79+30

NEW LEVEE ADJACENT TO EXISTING
SALT POND LEVEE
MUD DISPLACEMENT ALTERNATIVE

Project No. WOODWARD-CLYDE CONSULTANTS	Figure 2



* Note: Levee Materials As Described In Figure 1

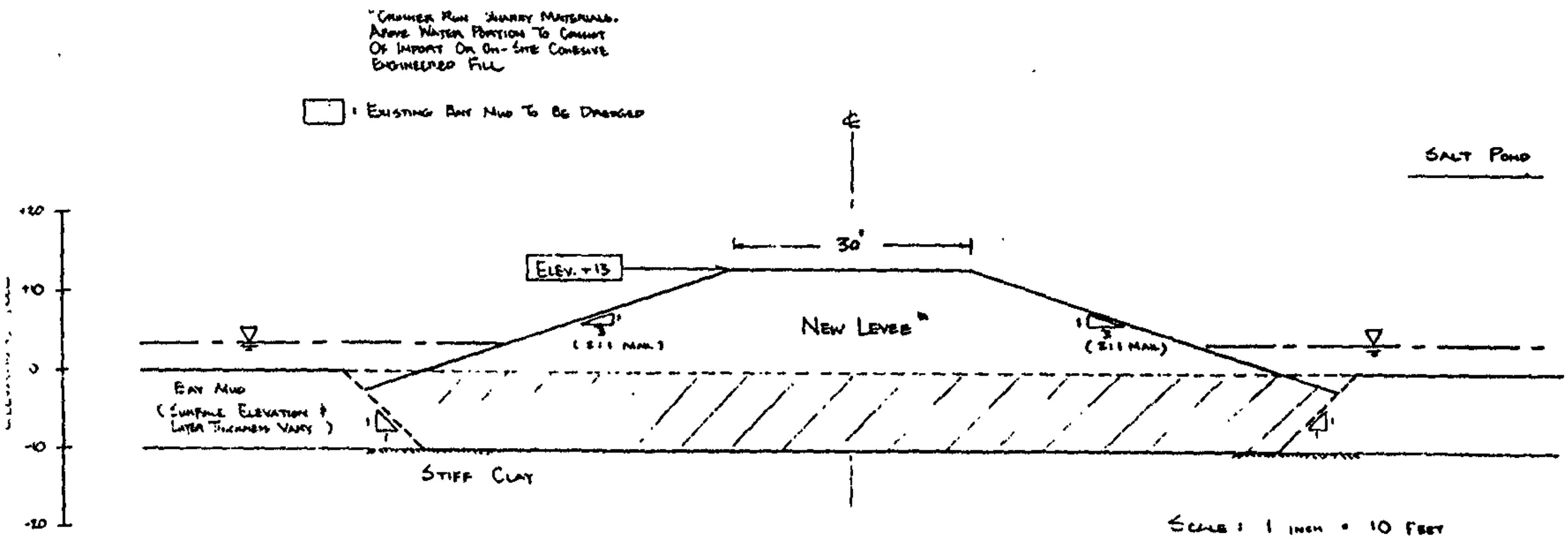
Scale: 1 inch = 10 Feet

Typical Section At Sta 79+30

NEW LEVEE ADJACENT TO EXISTING
SALT POND LEVEE
FLOATING EMBANKMENT ALTERNATIVE

Project No.
WOODWARD-CLYDE CONSULTANTS

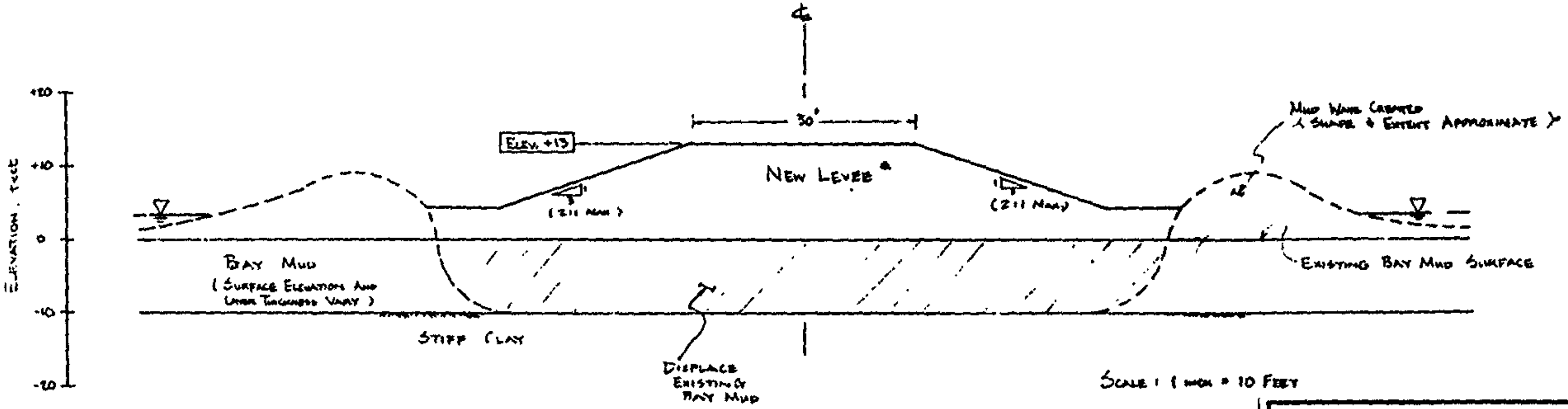
Figure 3



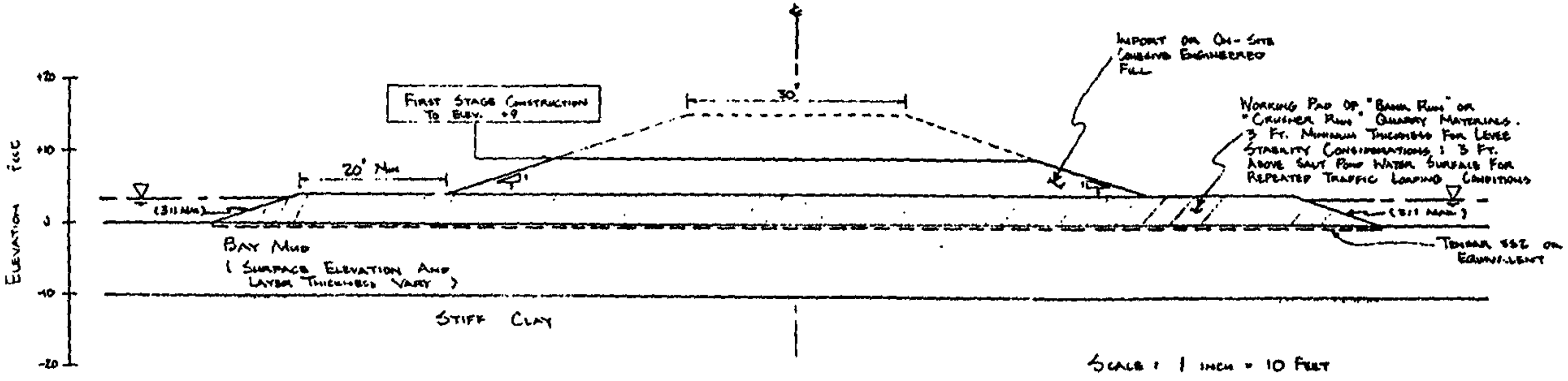
NEW LEVEE IN OPEN WATER PORTION
OF SALT POND
DREDGING ALTERNATIVE

Project No.
WOODWARD-CLYDE CONSULTANTS

Figure 4

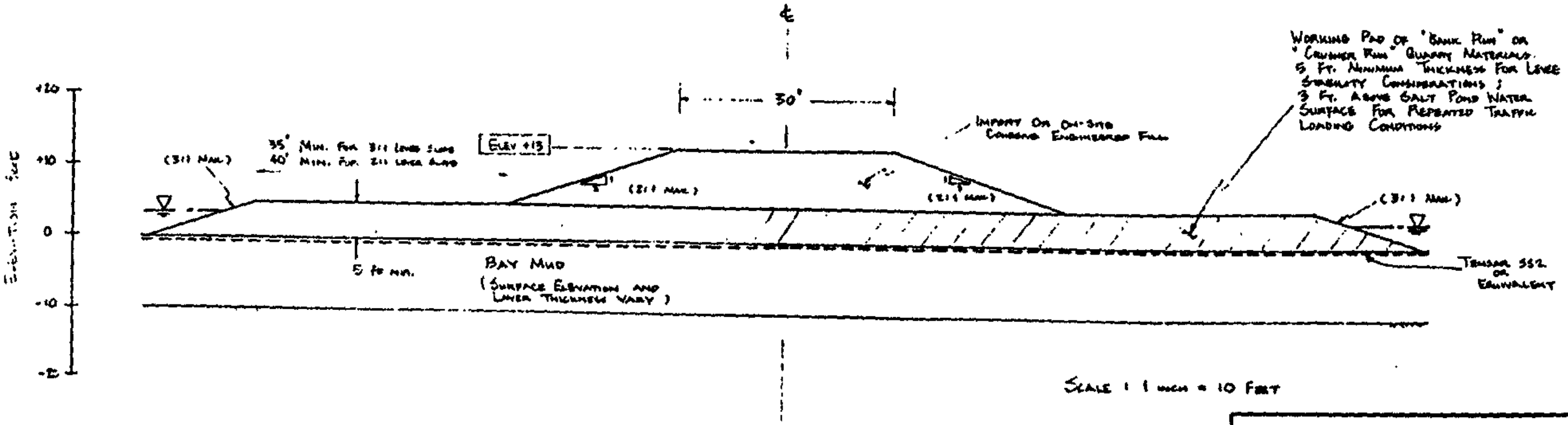


NEW LEVEE IN WATER PORTION OF SALT POND MUD DISPLACEMENT ALTERNATIVE	
Project No. WOODWARD-CLYDE CONSULTANTS	Figure 5


+20
+10
0
-10
-20
Elevation Feet
(3:1 Min)
20' Min
First Stage Construction To Elev. +9
30'
Import or On-Site Cohesive Engineered Fill
Working Pad of "Bank Run" or "Crusher Run" Quarry Materials. 3 Ft. Minimum Thickness For Levee Stability Considerations; 3 Ft. Above Salt Pond Water Surface For Repeated Traffic Loading Conditions
(3:1 Max)
Tensar SS2 or Equivalent
Bay Mud
(Surface Elevation And Layer Thickness Vary)
Stiff Clay
Scale: 1 inch = 10 Feet

NEW LEVEE IN OPEN WATER PORTION OF SALT POND STAGE CONSTRUCTION	
Project No. WOODWARD-CLYDE CONSULTANTS	Figure 7

Modifications Of The Floating Embankment Alternative Will Be Required In Deep Slough Areas


℄
30'
Elev +15
35' Min. For 3:1 Levee Slope
40' Min. For 2:1 Levee Slope
(3:1 Max.)
(2:1 Max.)
(2:1 Max.)
(3:1 Max.)
1
2
Import Or On-Site Cohesive Engineered Fill
Working Pad Of "Bank Run" Or "Crusher Run" Quarry Materials. 5 Ft. Minimum Thickness For Levee Stability Considerations; 3 Ft. Above Salt Pond Water Surface For Repeated Traffic Loading Conditions
Bay Mud
(Surface Elevation And Layer Thickness Vary)
5 ft min.
Tensar SS2 or Equivalent
Elevation, feet
+20
+10
0
-10
-20

Scale: 1 inch = 10 Feet

NEW LEVEE IN OPEN WATER PORTION
OF SALT POND
FLOATING EMBANKMENT ALTERNATIVE

Project No. WOODWARD-CLYDE CONSULTANTS	Figure 6